TRANSACTIONS

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1216

THE ST. CROIX RIVER BRIDGE.*

By C. A. P. Turner, M. Am. Soc. C. E.

The Wisconsin Central Railroad, now leased by the Soo Line, and forming the Chicago Division of that road, is being rapidly improved in order to offer to the traveling public service second to none between the Twin Cities and Chicago. The shortening of the line and the reduction of the grades by a number of changes in location are almost completed.

One of the most marked improvements is in that portion of the line near the St. Croix River, 7 miles above Stillwater, Minn. This new cut-off crosses the river at an elevation of 180 ft. above mean water level, and presents some novel and interesting engineering features. The bridge structure is 2 600 ft. in length. The river bed at this point is 1 800 ft. wide, between high sandstone bluffs, but the full width is overflowed only at high water. The material forming the bottom seems to be fairly uniform throughout, consisting of fine sand and silt, easily eroded, and extending down from 100 to 120 ft. to bed-rock.

The piers of the old bridge, ¾ mile below the new, are of masonry resting on wooden piles, cut off and capped with timber in the old-fashioned style. They seem to have been gradually shifting down stream, and to have moved from 6 to 8 in. from their original position at the top, as determined by observations from year to year.

One of the holes drilled to bed-rock, in the line of the new bridge, developed a flowing well, delivering, by Artesian pressure, a 2-in. stream of clear spring water 12 ft. above the river level.

* Presented at the meeting of December 6th, 1911.

With such conditions to meet, the problem of economic and satisfactory foundations required as careful consideration as that of the superstructure.

Selection of Type and Design.—When foundations are inexpensive, no type of structure has been evolved which is less in cost than the tower and girder trestle, and a design using 100-ft. girders between bents and 40-ft. girders over towers was examined, and this, in turn, was compared with trestle approaches and five 350-ft. arch spans, both in reinforced concrete and in structural steel.

The reinforced concrete design required more expensive foundations, and, for a single-track bridge, offered insufficient economy in the superstructure to offset such expense.

The superstructure of the trestle design could be built for nearly \$60 000 less than the arch design. Rip-rap for piers, on the other hand, would cost \$10 000 more for the trestle, due to the increased number of piers; piles would have cost \$25 000 more, and concrete \$60 000, leaving a net saving for the longer spans of approximately \$35 000.

A design intermediate in cost was submitted by the McClintic-Marshall Construction Company, using 200-ft. lattice girders between towers.

A further advantage possessed by the structure of longer span is the longer and wider piers and the decreased probability that they might be weakened by erosion. The box-girder top chords give a width of floor of 12 ft. from out to out, and 6 ft. 2 in. between flanges, which is more satisfactory than the floor of the ordinary girder bridge.

Foundations.—The trestle foundations and end abutments rest on sand rock. The four intermediate piers rest on piles, as at their location it is from 100 to 125 ft. to bed-rock.

The piles are of Norway pine, from 70 to 85 ft. in length. The penetration ranged from 50 to 80 ft. No grillage was used, the piles projecting about 4 ft. into the concrete mass of the pier. There are 110 piles in each pier. Most of them were driven by a steam hammer, and the time of driving ranged from 25 to 40 min. per pile. The penetration at the start was about 2 in. per blow, and, under the last blows of the hammer, it ranged from nothing to $\frac{1}{4}$ in.

The concrete for the piers was mixed on top of the bluff, on both the east and west sides, and was distributed as follows: A light timber

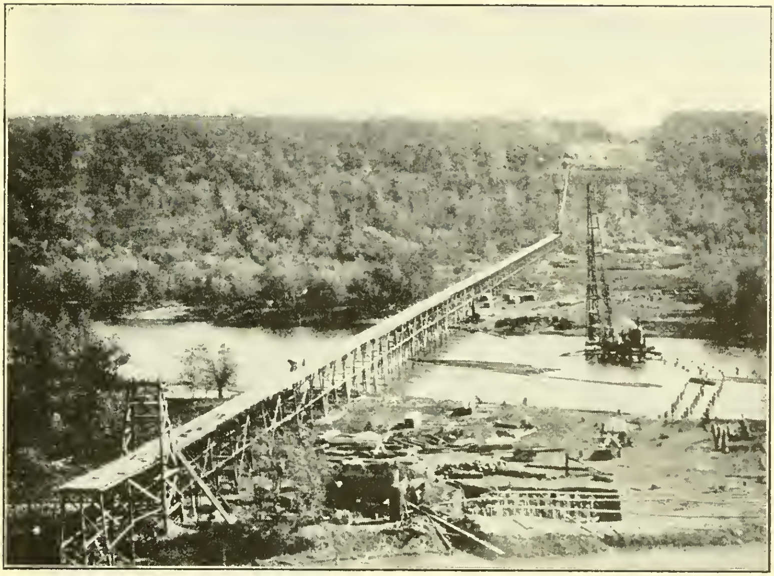

FIG. 1.—PILE-DRIVER, AND TRESTLE FOR HANDLING CONCRETE.

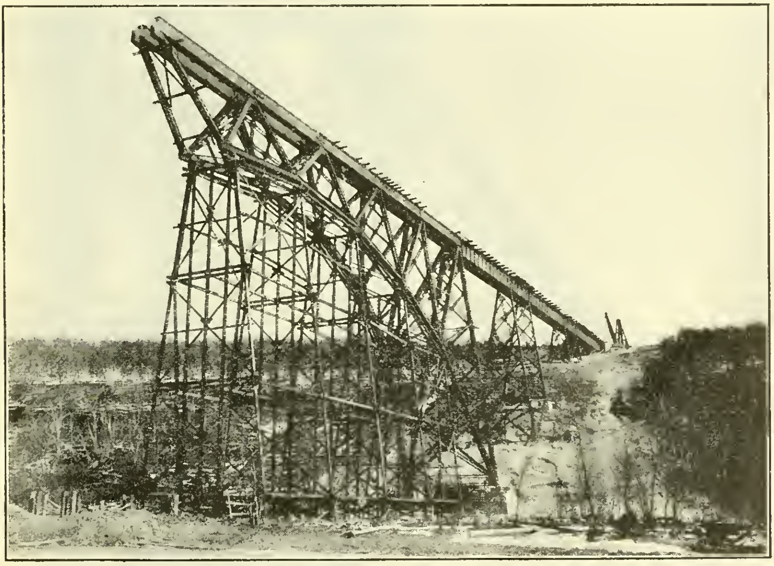

FIG. 2.—ERECTION OF EAST ARCH, ST. CROIX RIVER BRIDGE.

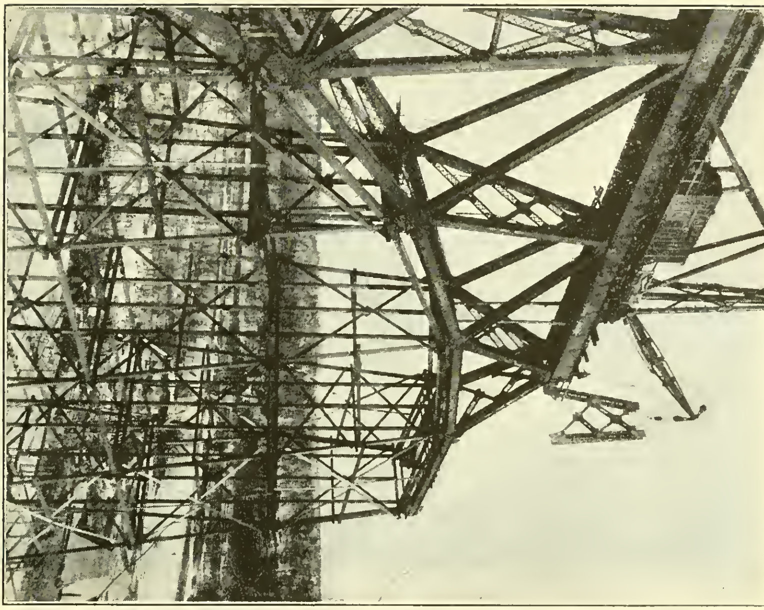

Fig. 3.—Erection of West Arch, St. Croix River Bridge.

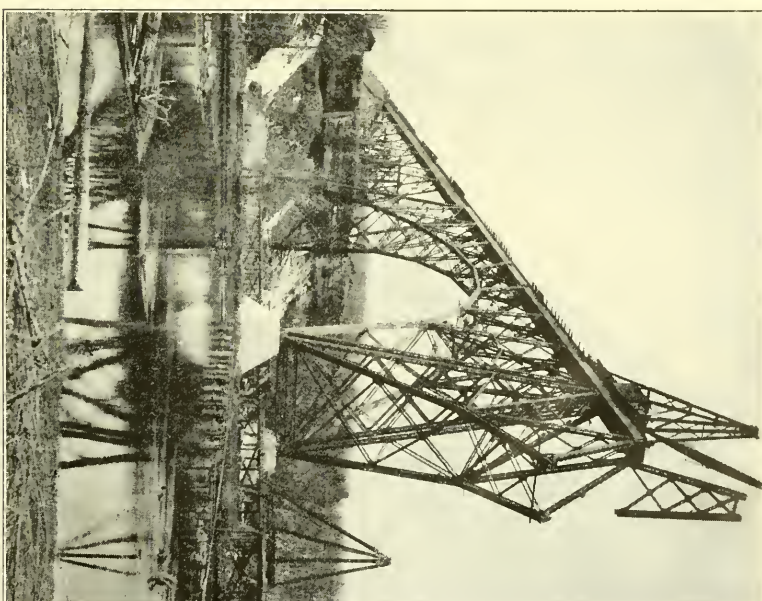

Fig. 4.—Erection of Half of Second Arch as a Cantilever.

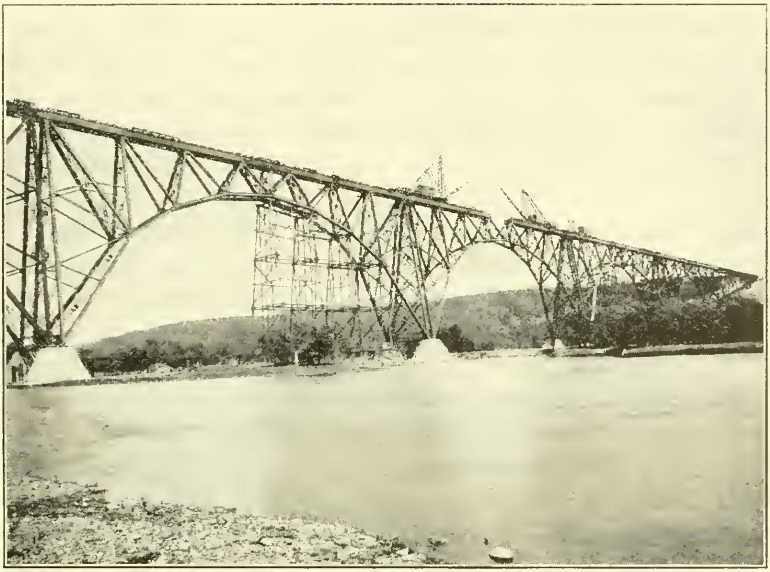

Fig. 5.—Erection of Center Arch Without Falsework, and the Half Falsework Used for Two Adjacent Spans.

Fig. 6.—The Closure of the Center Span.

trestle was constructed about 6 ft. higher than the top of the piers, and to one side. At the east end, the concrete was carried by a chute to a hopper and elevated sufficiently above the floor of the trestle to spout into a ½-yd. car. At the west end, a cableway carried the concrete from the mixer over the main channel to the end of the trestle, a distance of approximately 400 ft. Fig. 1 is a view of the trestle, cableway, and pile-driver.

Type of Span.—The type of span adopted was the three-hinged arch, with trusses battered 2 in. in 12. This inclination of the trusses renders them remarkably rigid in a lateral direction, and involves some skew details in the connections. Then, by a simple expedient, these were all reduced to simple bends.

For example, in the lateral connection for the bottom chord in the first panel beyond the shoe, the plate is bent centrally, about an axis transverse to the truss, to the same angle as that made by the intersection of the main chords projected on a vertical plane, while longitudinally there are two different bevel angles to meet the flange of the chords of different inclination. Slotting the plate, on the line of the transverse bend, to its intersection with the lines of longitudinal bend, leaves them simple bends, in place of die forgings.

The advantage of the three-hinged over the two-hinged or restrained type may be summarized as follows:

First.—The stresses are statically determinate, and provision is made for temperature stresses.

Second.—The frame adjusts itself to any inaccuracy of the foundation work, or slight settlement, without material change in the stress of the various members.

On the other hand, the ordinary three-hinged arch is inferior to the two-hinged in point of stiffness, and has generally been considered unsuitable for railroad traffic. The depth of many three-hinged arches has been made so attenuated on approaching the crown that the complaint concerning this type in that form is well founded.

In the design of the St. Croix River Bridge, the depth of the crown is 25 ft. and the rise is 124 ft., so that the truss has ample depth throughout.

In order to combine the advantages of two-hinged and three-hinged arches, as far as practicable, the connection of the central top chord was made in the form of a friction joint of five plates, two of bronze and three of steel, giving six friction surfaces. As soon as the live load comes on the central chord, these joints lock by friction as firmly as though riveted, and the frame then deports itself as a two-hinged arch.

This feature of the design, together with the inclination of the truss, renders the structure one of the most rigid in existence.

Erection.—The bridge was erected from both ends toward the center. The equipment consisted of four derrick cars, a lighter one at each end being used for unloading and sorting the material and bringing it to the end of the bridge, where it was handled by the heavier cars.

The end arches, throughout, were erected on falsework, which was placed by the derrick cars as the work was built out.

The most difficult part was the erection of the first panel of the end arches. On the east side, a falsework bent was put up, supporting the 100-ft. girder; and, on the west side, the end vertical of the arch was supported on a cob-pile of timber until the end bottom chord could be placed and the connection at the shoe made.

Fig. 2 shows half of the extreme eastern arch. Fig. 3 shows half of the west arch on falsework and the method of placing the frames. Fig. 4 shows the erection of half of the second arch as a cantilever, the other half being built on falsework. Figs. 5 and 6 show the closing of the fifth arch. This arch was erected without any falsework whatever.

Special Features.—In erecting past the center top chord, it was found simplest to support the center top chords from a timber bent down to the pin and to the outer end outside of the line of the chord, and slide it inward to position laterally after the succeeding chord and diagonals had been erected in place.

In building out the end arches, the compression of the timber falsework allowed the members to drop slightly below grade. The jacking up was done by taking advantage of the temperature changes between morning and noon. At about sunrise the shoe at the skew-back was wedged out; then, as the temperature increased, the expansion of the steel would raise the span from the falsework, and shim wedges would be driven home; then, as the temperature dropped, ½ or ¾ in. could be gained at the shoe.

As shown by Fig. 6, the skewback was arranged as follows: A cast-

PLATE I.
TRANS. AM. SOC. CIV. ENGRS.
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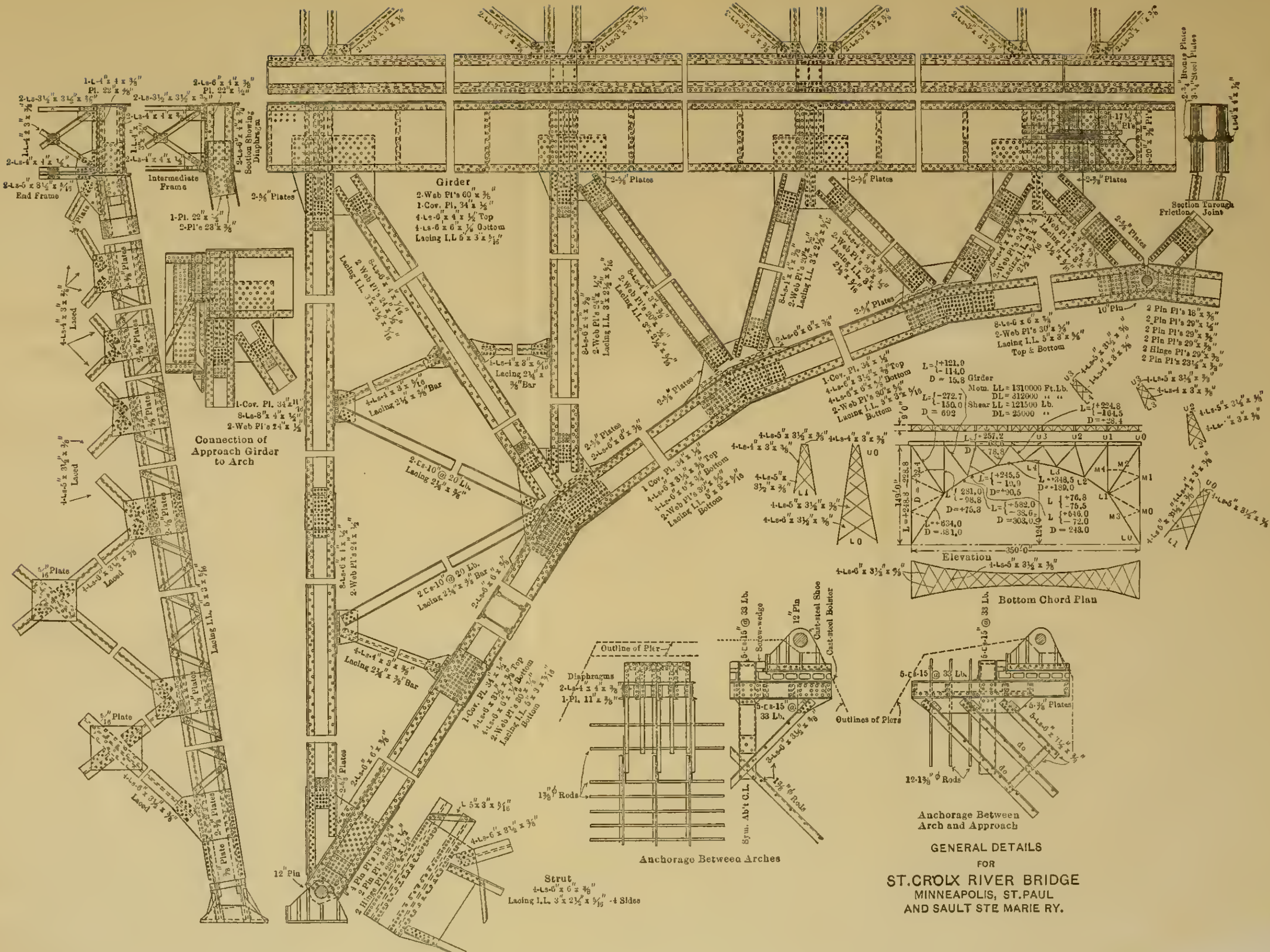
TURNER ON
ST. CROIX RIVER BRIDGE.



GENERAL DETAILS
FOR
ST. CROIX RIVER BRIDGE
MINNEAPOLIS, ST. PAUL
AND SAULT STE MARIE RY.

steel shoe was given a horizontal bearing on a bolster frame built into the concrete. This frame was arranged with a projecting vertical center member, against which the vertical end of the shoe could thrust. Steel blocking was provided between the vertical impost member and the end of the shoe, calculating for a clearance of about 9 in. Screw-wedges were provided for the adjustment of the span, so that this distance could be increased, to raise the crown of the arch, or decreased, if found to be above grade when erected.

In closing the first arches, erected wholly on falsework, this clearance proved none too great to get the shoe in place, and the scheme of jacking by temperature changes enabled the erectors to force it out so that the wedges could be entered.

The adoption of inclined trusses, with bent plate connections to the top chords, leaving the chord in a vertical position, is a somewhat unusual feature in bridge design.

The broad and exceptionally rigid floor system secured in the 50-ft. panels of the structure would seemingly warrant its adoption, from the economic standpoint, in deck bridges where the depth is not sufficient for arch construction. The elimination of floor-beams, floor-beam connections, and the combination of stringers and chords in one member, with ample width of floor, will generally result in a material saving of metal.

The structure was designed for Cooper's E-55 loading. Plate I gives a general idea of the details.

The work was executed under the direction of Mr. Thomas Greene, Chief Engineer of the Minneapolis, St. Paul and Sault Ste. Marie Railway, the writer acting as Consulting Bridge Engineer. The Kelly-Atkinson Construction Company was the contractor for the erection of the work, and the American Bridge Company furnished the steelwork. The concrete work was executed by the Railroad Company. The pile-driving was done by George W. Oakes and Company, of St. Paul, Minn.

Since this paper was prepared the writer has passed over the St. Croix Bridge several times in the cab of a heavy locomotive, and, from that point of observation, the bridge seemed to be the most rigid he had ever crossed.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1217

MULE-BACK RECONNAISSANCES.*

By William J. Millard, Jun. Am. Soc. C. E.

With Discussion by Messrs. George D. Snyder, Ernest McCullough, F. Lavis, and William J. Millard.

To engineers who have had experience with the mule, which possesses the intelligence of the horse and all the stubbornness of the burro, the title of this paper may seem somewhat ludicrous, but, with all his capriciousness, that animal has proved himself a capable and useful member of surveying parties in Central America. It may be that the writer has been absent from highly civilized countries too long to keep in touch with current literature, but he has never heard of the mule being used in the manner described in this paper.

Reconnaissance work, as far as the writer knows, has never been reduced to an exact science. On railroad work the old and experienced engineer is usually sent out to make the reconnaissance, and this he does in walking or riding over the country. By his experience he can tell whether or not a line is possible, as his eye is trained to measure cuts and fills, estimate the cost of bridges, and judge as to curvature, etc. In the United States he can usually obtain a map of the country from the Geological Survey, but in Central and South America, and in other countries which may be said to be in the first stages of development, maps are not obtainable, or are very inaccurate.

The railroad or mining engineer who is sent to the tropics to report on the feasibility of a railroad line or the opening of a mining concession, disembarks from the steamer at a little port, and is there

* Presented at the meeting of September 20th, 1911.

confronted by the wilderness. The company which has sent him out cannot allow him sufficient time to make a geodetic map of the country. He must get his information quickly and return home with his report. In all lines of business and in engineering there is a fixed relation between the degree of accuracy required and the work itself. In the reconnaissance survey the mule fixes that relation.

There should be two engineers in the party; one could do all the work, but two can observe more and make the work move more quickly. The instruments required are two small aneroid barometers, a pocket prismatic compass (or a plain pocket compass which may be read to 5°), a wrist strap holding a cheap watch reading to seconds, geological hammers, notebooks, pencils, etc. The work may be done with one barometer, but if two are carried it may prevent the delay caused by going back to pick up a known elevation if one barometer meets with an accident or has been jumped up a few hundred feet.

Naturally, the barometers carried should be capable of recording the highest elevation which will be observed, and they should be read to the nearest 2 ft. The geological hammers are necessary, as they aid in determining the character of the rocks, and this may be of great value to the railroad engineer. For example, certain rocks, when exposed to the elements, disintegrate easily and cause serious slides during the rainy season. Then, too, soft rock on the banks of rivers is likely to be eroded in the course of ten years, thus making it necessary to re-locate the line. In Central America the writer has seen the results of such mistakes in location, the cost of maintaining such a road being enormous.

Before leaving sea level both barometers should be read every half hour for several days. This can be commenced on the steamer while *en route*. In Central America the barometer reading is normal at 9 A. M. From that time until about 3 P. M. the readings will increase. This is illustrated by Table 1, which shows typical barometric readings in Guatemala City. At 7 A. M. the readings are 20 ft. low; at 9 A. M. they are normal; at noon they read 55 ft. high; and at 3 P. M. they have reached the maximum reading for the day.

At sea level the normal reading of Barometer No. 1 was 660 ft., that is, its constant was —660; and the constant of Barometer No. 2 was —310. Applying the necessary corrections, the figures in the columns headed "Barom. No. 1" and "Barom. No. 2" indicate the number of feet to be subtracted from readings taken throughout the day on the trail. Of course, areas of pressure in the atmosphere cause changes of several hundred feet, but by reading the barometers at the end of a day's journey and again before starting the next morning, the change can be detected. The writer's experience has been that when a heavy change occurs it takes 5 or 6 days to reach its maximum and about the same time to return to normal. It is not of very much consequence to catch these changes while the party is moving, as the difference rarely amounts to 30 ft. a day. As the average daily journey is 15 miles, and for much of the time is 25 miles, 1 or 2 ft. per mile would not be noticed. Using the table of corrections, the writer has checked the barometric heights very closely in a day's journey (within 10 ft. of the elevations printed on the time cards of some of the railroads in Central America). When an elevation of 5 000 ft. above sea level is reached, or when the party makes a change in location of 4° or 5° of latitude, it is well to make out a new table of corrections.

TABLE 1.—BAROMETRIC CORRECTIONS.

Time.	Corr.	Barom. No. 1.	Barom. No. 2.	Remarks.
7 A. M.	+ 20	— 640	— 290	Constant on No. 1 at sea level = — 660 ft.
7½	+ 15	— 645	— 295	
8	+ 10	— 650	— 300	
8½	+ 5	— 655	— 305	Ditto on No. 2 = — 310 ft.
9	0	— 660	— 310	
9½	— 5	— 665	— 315	
10	— 15	— 675	— 325	
10½	— 25	— 685	— 335	
11	— 40	— 700	— 350	
11½	— 50	— 710	— 360	
12 N.	— 55	— 715	— 365	
12½ P. M.	— 60	— 720	— 370	
1	— 65	— 725	— 375	
1½	— 70	— 730	— 380	
2	— 75	— 735	— 385	
2½	— 80	— 740	— 390	
3	— 85	— 745	— 395	
3½	— 85	— 745	— 395	
4	— 80	— 740	— 390	
4½	— 75	— 735	— 385	
5	— 70	— 730	— 380	
5½	— 65	— 725	— 375	

In order to obtain some idea of the rate at which a mule travels, it is advisable to ride him over a measured distance of 600 or 1 000 ft. at his natural walking gait. The writer's experience has been that the average mule travels $\frac{1}{20}$ mile per min. on ground varying from level up to a slope of 10° from the horizontal. On moderate hills he will make $\frac{1}{25}$, or possibly $\frac{3}{80}$, mile per min. On steep slopes he may make only $\frac{1}{40}$ mile per min., and on certain steep trails in Guatemala the writer has had occasion to allow only $\frac{1}{80}$ mile per min. Usually the rates of $\frac{1}{20}$, $\frac{1}{25}$, $\frac{3}{80}$, $\frac{1}{30}$, and $\frac{1}{40}$ mile per min. will suffice for the slopes generally met. The figure for jog trotting would be about $\frac{1}{12}$ mile per min.

It is the duty of the engineer who carries the watch to record the mule's rate, as indicated on the sample page of notes (Table 2). In order to approximate closely at first, it is well to select with the eye a point 100 ft. ahead (a stump, or tree, or clump of grass), and note the number of seconds required by the mule to traverse that distance. A table can be readily made, showing the number of seconds required to travel 100 ft. at the various rates, and can be kept at the back of the notebook. The time should be recorded whenever the rate changes.

The time is also noted when certain prominent topographic features are passed, and when the course is changed, but not closer than $\frac{1}{4}$ min. The notes and explanations on Fig. 1, a sample page of the notebook, illustrate the method of carrying on the work. The notes are recorded in lead pencil by the engineer who rates the mules. On the left page (Table 2) he records the time, rate, azimuth or compass course, and the readings of the barometers. On the right page (Fig. 1) he sketches the country as far as he can see, and also makes note of the geology. Notes regarding the compass and the geology are made by the second engineer. Whenever he calls out a change in the course, the first engineer records it, together with the time. Of course, the notes are very crude and difficult to make out, and therefore they must be put in order and inked in immediately after stopping work each day.

The sample notes show that Engineers Dawson and Millard left the hotel at San Felipe at 7.31 A. M., going at the rate of $\frac{1}{20}$ mile per min. The barometric elevations were compared, and checked within 10 ft. The course taken read 20° on the compass. At 7.33 A. M. the course changed to 10°, and at 7.33$\frac{3}{4}$ the last street of the village was passed. At 7.37 the grade became steep enough to change the rate to $\frac{3}{80}$ mile per min., and at the same time the course was changed to 0°, etc., etc.

The figures in the column headed "Dist." represent the fractions of miles traveled on each course, and are deduced for the purpose of plotting the notes. The corrected barometric readings are written in red ink (in italics in the sample reproduced).

TABLE 2.—RECONNAISSANCE, SAN FELIPE TO QUEZALTENANGO.

Mule back { Dawson. Millard. Oct. 10, 1909.

Time.	Rate.	Dist.	Azimuth.	Barom. No. 1.	Barom. No. 2.
33¾	....	....	270°	3340 *2685*	3000 *2685*
33	1/20	$\frac{3/4}{20}$	....	....	....
26½	1/30	$\frac{6\frac{1}{2}}{30}$	310°	....	....
23	Started.				
22	Stopped.				
21	1/40	$\frac{6\frac{1}{2}}{40}$	315°	....	....
18¼	....	$\frac{2\frac{3}{4}}{25}$	300°	....	....
15½	....	$\frac{2\frac{3}{4}}{25}$	275°	....	....
10	....	$\frac{5\frac{1}{2}}{25}$	290°	....	....
6¾	....	$\frac{3\frac{1}{4}}{25}$	310°	....	....
4	....	$\frac{2\frac{3}{4}}{25}$	325°	....	....
2¼	....	$\frac{1\frac{3}{4}}{25}$	330°	3375 *2725*	....
8 A. M.	....	$\frac{2\frac{1}{4}}{25}$	300°	....	....
56	1/25	$\frac{4}{25}$	....	....	....
53½	1/30	$\frac{5}{60}$	320°	....	....
46	1/20	$\frac{15}{40}$	0°	....	....
41	....	$\frac{15}{80}$	345°	....	....
37½	....	$\frac{21}{160}$	350°	....	....
37	3/80	$\frac{3}{160}$	0°	....	....
33	....	$\frac{4}{20}$	10°	....	....
7.31 A. M.	1/20	$\frac{2}{20}$	20°	3165 *2520*	2815 *2530*

The accuracy of this method of surveying is surprising. The writer has had the opportunity to check his work with geographical maps, and found no discrepancy when plotted on the scale of 1 mile to the inch. On a reconnaissance from Quezaltenango to Chiantla, in Guatemala, and back to Quezaltenango, a circuit of 81 miles was made. For six hours on the first day out it rained so hard that it penetrated the oil-skin slickers worn by the party. The trail was very mountainous, and the $\frac{1}{80}$-rate was used frequently. The error in closing was only $\frac{3}{8}$ mile, and fell within the limits of the town site. The compass read-

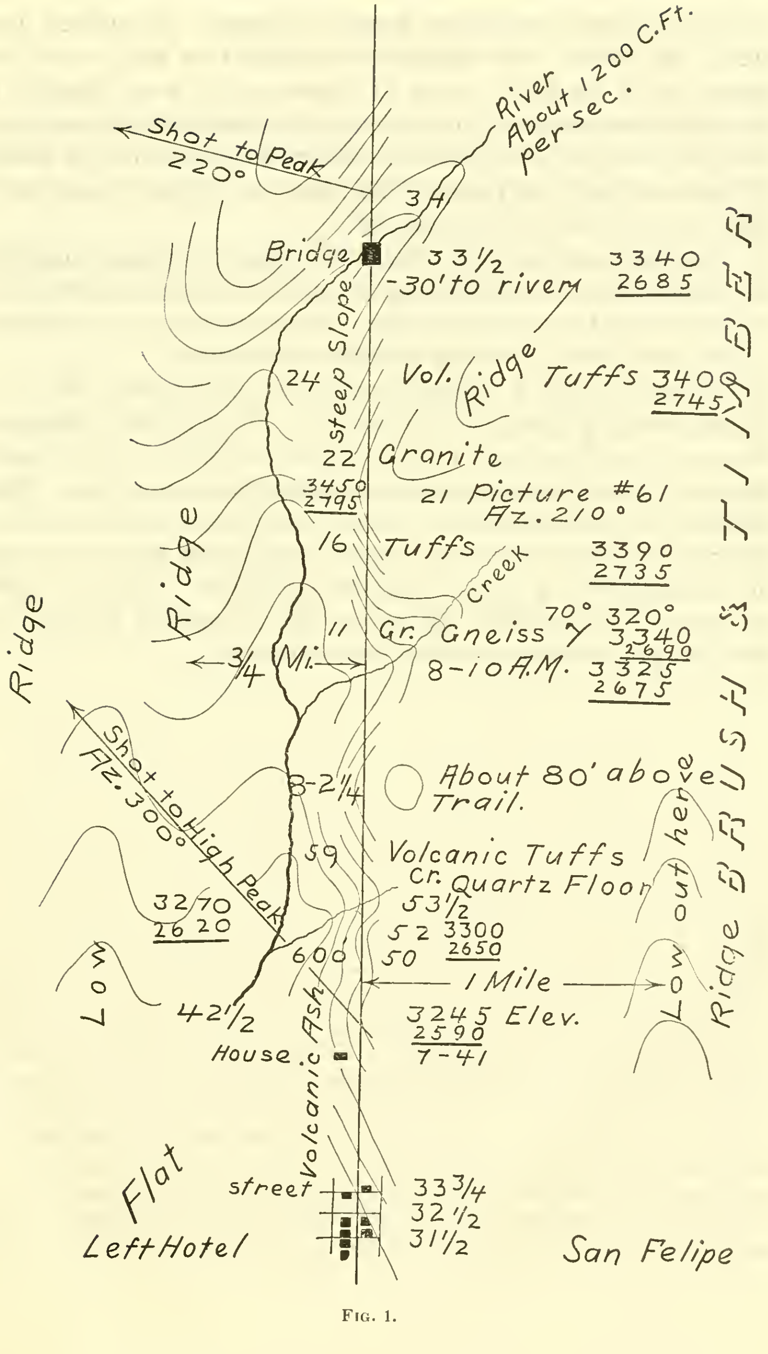

River
About 1200 C.Ft.
per sec.
Shot to Peak
220°
34
Bridge
33½
-30' to river
3340
2685
steep Slope
24
Vol.
Ridge
Tuffs 3400
2745
22
Granite
3450
2795
21 Picture #61
Az. 210°
16
Tuffs
Creek
3390
2735
Ridge
Ridge
11
Gr. Gneiss 70° 320°
3340
2690
3/4 Mi.
8-10 A.M.
3325
2675
Shot to High Peak
Az. 300°
8-2¼
About 80' above
Trail.
59
Volcanic Tuffs
Cr. Quartz Floor
53½
3270
2620
52 3300
2650
600
50
1 Mile
Low
42½
Volcanic Ash
3245 Elev.
2590
7-41
House
Low out here
Ridge
Flat
street
33¾
32½
31½
Left Hotel
San Felipe

Fig. 1.

ings were seldom taken closer than to 5 degrees. In addition, the survey was plotted with topography extending to 4 and 5 miles on either side of the trail. Owing to the scarcity of heavy brush and timber in Guatemala, the topography could be sketched very well, but, even in Nicaragua, where the brush is very thick, shots may be taken to prominent peaks and ranges of hills, and, when plotted, furnish data much more reliable than the memory.

The crude map thus produced certainly helps the railroad engineer to eliminate routes for preliminary surveys, which might otherwise be undertaken, and the mining engineer can easily reduce to order the mining, prospecting, and other geological information.

The cost of such a reconnaissance is not greater than that of a survey depending on eye and memory only. The party must penetrate the country, and whether it travels by canoe, on foot, or on mule back, the cost of taking notes in the form described herein is nothing. The information obtained, however, may be such as to warrant discontinuing all work, thereby saving money which might have been spent on further surveys of a more substantial character. Under the right conditions, it is essentially the survey for the engineer who has in mind the proper relation between time and money.

DISCUSSION

George D. Snyder, M. Am. Soc. C. E. (by letter).—This paper is of value in calling attention to methods of reconnaissance and mapping which are too little known. Although there is very little civil engineering literature on this subject, it is treated quite exhaustively in works on military topography and sketching. The art of hasty mapping has reached a high state of development by army officers, because, under the necessities of a campaign, it is of more importance to have a map which indicates every road, path, and trail, even if it has errors in distances of from 5 to 10%, than one which has no perceptible errors, but is not up to date and does not show every road. Mr. Snyder.

In common with all mapping, such work is divided into methods of measuring distance, direction, and elevation. The aneroid barometer for elevation and the compass for direction are used almost invariably, but there is a great variety of methods of measuring distance: by timing a horse or mule, after obtaining his rate, as described in this paper; by counting the topographer's paces, or by using a pedometer or passometer; or, where a wheeled vehicle can be used, by counting the revolutions, or using a cyclometer, or odometer; or, on the water, by the use of a patent log, or by counting the revolutions of the screw or wheel; and very accurate results can be obtained with a bicycle.

It is not always the case that where roads exist suitable maps can be had, and, even where accurate maps are available, there is often lacking special information which is necessary to the object of the reconnaissance and can be added by methods similar to those described.

Mr. Millard's method is to record his observations in a notebook and from them prepare a map. What is perhaps a preferable method under most conditions, and will save some time, is to use one of the several forms of sketching cases now on the market, which are nothing more or less than hand plane-tables with a compass for orientation. These devices enable one to plot the route traversed and sketch the topographical features directly on the map, thus saving time and avoiding errors which would not be obvious in a notebook, but are apparent if placed on a map. It is surprising how quickly and accurately an area can be mapped by such methods; and the value of such a rough map, as a preliminary to more accurate surveys, is great, for, as stated by Mr. Millard, it may often furnish information which will indicate that more accurate and expensive surveys are not warranted.

A reconnaissance is made for most engineering projects, and notes of the region traversed are taken in a more or less disjointed manner. Instead of this, a skillful man, in practically the same time, can obtain a map indicating the controlling topographical features, Mr. Snyder. with distances to within 5%, and other notes of value bearing on the project in hand.

Mr. McCullough. ERNEST MCCULLOUGH, M. AM. SOC. C. E. (by letter).—As Mr. Millard states that he had never heard of a mule being used for reconnaissance work, the writer would like to add a note of historical interest. During the winter of 1889-90 the writer was employed to make a large topographical map of a number of mines and mining claims in California. The engineer who made the survey, and who had been employed for several months on the work, having lamed his right hand, turned the notes over to him.

The work involved the running of a closed traverse to tie in the claim corners and all the more important features. When this had been done, a "filling-in" survey was made, in order to supply the topographical data, using a mule. A saddle with a high pommel had attached to it a cheap clock and a compass hung in gimbals. A sheet of paper, properly ruled, was fastened to a board strapped to the left arm, and an aneroid barometer was hung by a strap over the shoulder. The mule walked along the trails, and at 15-min. intervals the time was read and also the bearing of the compass and the height as indicated by the aneroid. The sheet of paper was ruled horizontally into 15-min. intervals and vertically into 9 columns, each varying 10 degrees. The compass was graduated from 0 to 360°, and was read only to the nearest degree by the observer, who put down the reading in the column headed by the nearest lower angle. This was done merely for convenience, and the barometer reading was placed on the same horizontal line just ahead.

Every line was numbered. On the sketching board was placed another sheet of paper, ruled in ten squares to an inch. If the observer thought sketches would help out at any point, he made them on the quadrille-ruled paper, placing on each sketch the number of the line indicating the point at which it had been made. The sketch resembled that shown by Mr. Millard, the same principles being followed. The map was made on a scale of 2 in. to the mile, and the "filling-in" notes closed remarkably. There were many trails through that section, and the mule was ridden over nearly all of them. Vertical angles were taken on the traverse survey, and all elevations were computed from them, so that the aneroid readings were checked whenever possible. In 1890 the writer made a similar survey, and found the method very good for the purpose for which it was used. At that time (22 years ago), it was not considered a new idea, but was thought to be of considerable antiquity.

Mr. Lavis. F. LAVIS, M. AM. SOC. C. E.—This paper is interesting in so far as it describes a method of "filling in" topography which may be quite useful for certain purposes. The idea is by no means new, but the

Mr. Lavis.

details are described more fully than has been done elsewhere.* If the author had been content to limit its application to this filling in of topography on maps to be drawn to a small scale, the prominent features of which had been controlled by triangulation or other adequate methods, or as a means of locating roughly certain broad general geological features, there would be little criticism possible, but the speaker is of the opinion that its utility, in connection with the location, or any investigation into the feasibility or practicability, of proposed lines of railway, is questionable, inasmuch as it is likely to lead to the assumption that certain information has been obtained which in reality has not. If in what follows the speaker offers some criticism, it should be understood to be solely of the author's proposition to apply his methods to railroad reconnaissance and not of the methods themselves. Referring to reconnaissance, the late A. M. Wellington, M. Am. Soc. C. E.,† wrote, more than a quarter of a century ago, as follows:

"The feeling should always be present in the mind of the engineer that he ought to be somewhere over the edge of the horizon, or on the other side of the valley or ridge, instead of following his nose where he is," and again: "We may *survey* lines, but we must never reconnoitre them. If we do, it is not a reconnaissance."

He insisted again and again that the reconnaissance should be of an area and not of a line, but the true significance of this does not yet seem to be properly appreciated.

The author's conception of the duties of the railroad engineer who is sent to report on the feasibility or otherwise of a proposed line of railway, does not seem to be adequate. It is difficult to imagine a man having the broad outlook suggested by Wellington, while counting the steps of a mule. The engineer who is to make such an examination should be one who is not only experienced in the location of railroads, but also in their construction and operation, especially the latter; he should have some knowledge of the methods of financing such propositions; of the information wanted by bankers or financiers, and of how this should be presented to them. He should pay especial attention to the amount and nature of the business which will come to the road, as the kind of road which should be built will depend on this. Such an engineer, as he rides over the country, will pay far more attention to the probable business of the road than to the gait of his mule, or even the distance or direction in which he is traveling. This may seem to be rank heresy to those who have thought of the pioneer railroad engineer, with his barometer and compass, as the legitimate successor of the great explorers in opening up undiscovered countries, but, prosaic as it is, it is a fact.

* "Railroad Location Surveys and Estimates," p. 18.

† "The Economic Theory of Railway Location," pp. 835 and 837.

Mr. Lavis. Many engineers and contractors of sufficient experience can, by riding over a route, make a fairly close estimate of the probable cost, that is, the grading and bridging. Contrary to Mr. Millard's ideas, however, they are not "trained to measure cuts and fills, estimate the cost of bridges, and judge as to curvature, etc.," by eye. They very carefully refrain from going into details of this nature without a survey, but, as the result of long experience on many lines in different kinds of country, they can estimate in round figures how much per mile certain lines or sections of lines will run for the grading and bridging; and, as this represents usually only about 50% or less of the total cost of the complete line with equipment, the other 50% being for items, the cost of which can be determined quite accurately with little reference to topographical conditions, any variation in the total cost is usually well within the allowable limits for preliminary estimates.

It is usually necessary, however, in addition to an estimate of the cost, that the engineer should know fairly closely the length of the proposed line and have some adequate idea of its physical characteristics, that is, the approximate ruling grade, amount of rise and fall, and amount and character of curvature, the first being the most important and most difficult to estimate.

The fact that Wellington and other writers have stated that it is possible for the experienced locator to eliminate certain manifestly impossible routes simply by looking them over, has been misinterpreted quite generally, and there seems to be a feeling that, by some occult means, a close approximation of these physical characteristics may or should be made without going to the cost and trouble of making an instrumental survey. Where topographical maps, equal to those of the United States Geological Survey, are available, this may be done by projecting the proposed lines on these maps; failing this, there is, in the speaker's opinion, no recourse except in some cases in easy country, and even then it is well to be very careful, and to make, at the very least, a rough skeleton stadia survey.

In a former discussion on this subject, S. Whinery, M. Am. Soc. C. E.,* referred to: The Genius * * * who, disdaining plats [and presumably transits, levels, etc.], takes in at a glance the P. C., and the degree of a curve that will fit the hill comfortably, and, at the proper point on that curve, sails off on a tangent that will strike the bull's eye 2 miles away. Such geniuses, however, are apparently and fortunately all gone, and the genius nowadays, who is not nearly as spectacular, spends his time, in his tent at night, wiggling a piece of black thread back and forth on his map, figuring how he can do away with the curve and not spend any more money, and he generally does it.

* *Transactions*, Am. Soc. C. E., Vol. LIV, p. 143.

The author evidently recognizes the difficulty of guessing by eye at either the length of line or the rate of grade in such country as that between San Felipe and Quezaltenango in the "Altos" of Guatemala, which can only be described as being "all on end" and "up in the air." He is mistaken, however, in assuming that there is, or at least should be, any excuse for guessing at all in such country. It is not necessary to make a geodetic survey for any purpose whatever in connection with railroad location, but the engineer who is asked to make a reconnaissance in such country—or in any other where it is necessary to supplement the "eye" of an ordinarily well-trained engineer—should either insist on sufficient assistance and time for an adequate survey or refuse to give his opinion. Mr. Lavis.

The author states that two men are necessary to carry out properly the methods described by him, and that they average about 15 miles a day in rough country. Two good men and a native with a stadia can cover from 5 to 7 miles, and they would obtain information which would be of real value. The survey may be ever so rough and sketchy, but it does give actual distances and elevations correctly within the requirements of the case.

Despite all that has been written as to their value, the speaker regards the aneroid and the compass, for use in connection with railroad reconnaissance, as little better than toys. It is seldom that the railroad engineer has any opportunity to compare aneroid readings with those of a mercurial barometer at a base station, and here again, with instruments delicate enough to give sufficiently close readings, assistants to read them, and methods of obtaining distances and directions sufficiently accurate to determine, even approximately, the possible rate of grade on a long supported line. The expense incurred and the time consumed in such observations would be very little, if any, less than would be required to run a rough stadia line which would give so much more and better information of the kind wanted by the railroad engineer.

The only information which is of use to him, in order that he may determine the physical characteristics of a line, even approximately, must give distance, elevation, and direction with a fair degree of accuracy. It is the speaker's opinion that the methods proposed by the author will not do this. A precise survey is not necessary, but a stadia survey, intelligently made, will give the information, and its expense will be very little if any greater than would be incurred in carrying out the methods proposed.

One cannot doubt the author's statement that he closed a traverse of 81 miles in the extremely rough country referred to, within $\frac{3}{8}$ mile, by estimating the rate the mule walked and reading the compass to the nearest 5°, though this would almost be a fair check for a rough

Mr. Lavis. stadia line, and great credit should be given to the intelligent mule who was such a capable and useful member of the party.

The speaker thinks that the picture of tropical America drawn by the author is more impressionistic than real. He says:

"The railroad or mining engineer who is sent to the tropics * * * disembarks from the steamer at a little port, and is there confronted by the wilderness."

The reader's imagination will picture the whole of America, from the United States to Buenos Aires as a wilderness of tropical vegetation, dotted here and there along the coast by "little ports" with a few thatched huts and cocoanut palms. He may forget that Havana, Vera Cruz, Panama, Cartagena, Rio de Janeiro, Lima, and a dozen other cities are ports of the tropics, and that in the interior there are a number of important and interesting cities, many of whose inhabitants are educated and cultured people.

The entrances to Guatemala, it is true, are not particularly impressive, but they are all connected by railroads with the interior, and three of the four ports are connected by rail with the Capital. From Puerto Barrios, on the Atlantic side, there is an excellent line of railway, with track fairly well ballasted, steel bridges, and modern rolling stock, running to Guatemala City, where connection is made with a line running to San José and Champerico, ports on the Pacific coast, and which very shortly will be connected with the National Railways of Mexico, giving all-rail communication with New York. From the Atlantic coast, the journey to the Capital may be made in reasonable time in a comfortable chair car with an observation platform. Leaving Puerto Barrios, instead of a wilderness, the line passes for 60 miles through banana plantations, from which nearly 1 000 000 bunches will probably be shipped to the United States this year. Beyond Guatemala City, the Pacific slope is covered with coffee and sugar plantations, the coffee production alone amounting to some 40 000 tons per annum, valued at not less than $8 000 000 in gold.

Next below Guatemala is Salvador, with a population of more than 1 000 000, on an area of about 7 800 sq. miles, averaging nearly three times as many per square mile as in the United States. One may ride for days and see little land that is not under cultivation, and it is probable that, in a few years, the existing lines of railway will be connected with each other and with the railways of Guatemala, and so with Mexico and the United States.

There are many large areas in the United States itself which will not compare particularly favorably, either in exploitation of natural resources or in the culture and hospitality of the inhabitants, with many of these countries. What has been noted in regard to Guatemala and Salvador will apply in greater or less degree—and on the whole it will be greater rather than less—to all the other countries of tropical America, and it is hardly fair, even unintentionally, as was no doubt the case in this instance, to convey the impression that Latin America is a wilderness, waiting for the exploring engineer and his mule. Mr. Lavis.

WILLIAM J. MILLARD, ASSOC. M. AM. SOC. C. E. (by letter).—The criticism of this paper is mostly from the standpoint of an engineer on preliminary and location surveys who fears that his field is being invaded. The impression that the mule is going to supplant a locating party, and that his rider will distribute P. C.'s and P. T.'s around the hills, is rather erroneous. Mr. Millard.

Let it not be forgotten that reconnaissance work belongs to other branches of engineering as well. It is quite customary to-day for large tracts of land to be granted in concession to individuals or companies engaged in commerce, agriculture, or mining and prospecting. Most of these grants are made by small States or by colonial governments which desire to attract capital and open up their country, and the greater part are located in the scarcely known and unmapped portions of the world. A promoter is often behind these various concessions. He paints a glorious picture of the agricultural, commercial, or mineral advantages which may be investigated, or caused to be investigated, by somebody who wants to buy. These concessions may comprise tracts of from 20 to 20 000 sq. miles. The engineer leaves New York accompanied by perhaps two assistants and armed with a kodak; he returns from 3 to 5 months later with a rough map of the territory, photographs, and sufficient data to demonstrate what is best to do.

While the time of option is usually very limited, there may also be climatic obstacles. For example, in the eastern part of Nicaragua the dry season lasts from 2 to 3 months. In the wet season the trails are absolutely impassable. The mud is knee-deep and neither man, mule, nor ox can travel for any distance. In one of these dry seasons, the writer was one of a party of three who made a rough map of 600 sq. miles of mining territory, including a run of 114 miles to the coast, to see whether a railroad was "possible." If the work had not been done rapidly, it would have been necessary to wait another year.

A barometer is not an exact instrument, but its daily variation is easily tabulated. The variation due to a "high" or "low" area will not be noticed if spread over a week's traveling of from 10 to 30 miles per day. The barometer is a very valuable and scientific "toy." If it is supplanted by a transit or a level, the survey can no longer be called a mere reconnaissance. In railroad work, the use of these instruments indicates a preliminary, as far as exploration work is concerned.

Reconnaissance work is the rough surveying of areas. If the country is open and clear, one may obtain better results and more

Mr. Millard. quickly than when it is densely wooded or covered with jungle. In mule-back work the trails and accessory territory are plotted at night with interpolations regarding territory not seen.

Reference was made to the progress obtained by using Indians with stadia rods and a hand-level stadia. A progress of 7 miles a day in an open country cannot be compared to from 20 to 30 miles, although the information is more exact. However, in a country like the eastern part of Nicaragua, it would be absolutely impossible, except on the narrow savannah bordering the coast, to use the stadia hand-level. The Indian would literally have to cut his way through the tangled growth of vines, trees, and brush. For reconnaissance work this method is too costly. The ordinary hand-level stadia cannot be read at a distance of more than 500 ft., and the writer is quite sure the limit is 300 ft.

A reconnaissance is made strictly according to the definition of the word. It is not accurate. It is rough, and quickly made, but it must be supplied with copious notes. Naturally, if these are taken by a third member who is capable of figuring (as for a railroad) tonnage for the future, etc., the notes will be as valuable as it is possible for them to be.

Mule-back methods are not the only ones by which reconnaissance work may be done; but mule-back methods do mean a saving of time and money, whether the country surveyed is high, open, low, or covered with brush and timber. It is a method that brings results. Too many parties, leaving the United States for foreign parts, have spent most of their time in building camps and using painstaking methods on a survey which was more adapted to Broadway than to the wilds.

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TRANSACTIONS

Paper No. 1218

CONSTRUCTION OF THE MORENA ROCK FILL DAM, SAN DIEGO COUNTY, CALIFORNIA.*

By M. M. O'Shaughnessy, M. Am. Soc. C. E.

With Discussion by Messrs. George L. Dillman, George F. Maddock, J. D. Galloway, H. Hawgood, F. B. Maltby, and M. M. O'Shaughnessy.

In August, 1906, under a contract with the City of San Diego, the Southern California Mountain Water Company undertook to furnish a supply of water, up to 7 766 000 gal. per day, at the city limits, at the extraordinarily low price of 4 cents per 1 000 gal., from the Lower and Upper Otay Lakes, through 20 miles of pipe line.

Previous to this time the city depended on the pumping plant at the San Diego River, in the Mission Valley, inside the city limits, for its supply. As this water was highly alkaline and the quantity was limited to about 3 000 000 gal. per day, the city was very eager to secure an improved water supply of good quality to supplant the objectionable system, which retarded the growth of the community.

As the volume of water in the Otay water-shed alone was not sufficient to fill the obligations of this contract, the Company undertook to expand its plant and make additions to provide for future increased consumption. For this purpose the Dulzura Conduit was built, in 1907-08, to lead the water from the Cottonwood water-shed

* Presented at the meeting of December 20th, 1911.

into that of the Otay, and the construction of the Morena Dam was proceeded with, on the completion of which it was intended to impound the flood-waters of the Cottonwood and gradually pass them into the Otay as required. Table 1 shows the condition of the present system (Fig. 1).

TABLE 1.—RESERVOIR SUPPLIES OF THE SOUTHERN CALIFORNIA MOUNTAIN WATER COMPANY.

Name of reservoir.	Type of dam.	Height, in feet.	Outlet altitude, in feet.	Area submerged, acres.	Capacity, in gallons.
Lower Otay.......	Rock fill...............	150	400	869	13 000 000 000
Upper Otay.......	Arched concrete......	77	521	164	1 090 000 000
Chollas Heights...	Earth and steel plate..	34	385	17	90 000 000
Barrett............	Proposed, not designed	150	1 506	936	15 000 000 000
Morena............	Rock fill...............	265	2 912	1 370	15 000 000 000
				Total capacity................	44 180 000 000

SOUTHERN CALIFORNIA MOUNTAIN WATER COMPANY'S PIPE LINE AND RESERVOIRS

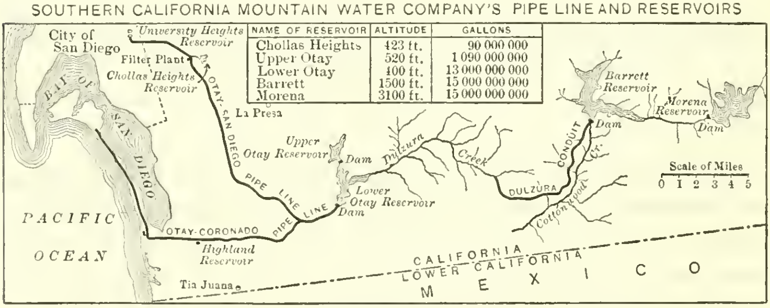


FIG. 1.

In this part of California it is necessary to store enough water to provide for the consumption for a period of from 5 to 7 successive dry years. The run-off generally takes place from January to May, and the stream beds are dry for the remainder of the year.

The Company's works are on Otay Creek and the Cottonwood River, about 25 and 50 miles, respectively, southeast of San Diego. The Cottonwood water-shed has an area of 250 sq. miles above the Barrett Dam and the Dulzura Conduit Intake—135 sq. miles of which are above the Morena Dam. The Otay water-shed has an area of 100 sq. miles, so that the Company commands altogether 350 sq. miles of clean mountain water-shed covered with wild brush. There are practically no traces of animal life in this region, therefore the quality of the water is perfect. On account of the purity of the water furnished, San Diego, at the present time, has the lowest death rate in the State of California, and is absolutely free from typhoid, except for cases imported from outside districts.

Rainfall and Run-off.—Prior to the last five years, no records were kept of the rainfall or run-off on the Company's properties. Since that time, however, rain gauges and weirs have been established, and daily records have been kept, from which Table 2 has been compiled.

TABLE 2.—RAINFALL.

Year.	Barrett Dam rain gauge. Elevation, 1 700 ft.	Morena Dam rain gauge. Elevation, 3 300 ft.	Run-off, in gallons, from Cottonwood water-shed at Barrett, 250 sq. miles area.
1906........	29.94 in.	34.73 in.	19 506 000 000
1907........	12.79 "	18.56 "	11 080 000 000
1908........	16.82 "	20.56 "	4 227 000 000
1909........	24.54 "	32.98 "	9 414 000 000
1910........	11.28 "	13.94 "	5 500 000 000

Table 3 shows the catchment on the water-shed adjoining to the north, which supplies the Sweetwater Dam, and on which accurate records have been kept for more than 20 years.

TABLE 3.—RUN-OFF, SWEETWATER WATER-SHED.

Season.	Rainfall, in inches, for each season at Sweet-water Dam.	Run-off, in gallons.	Season.	Rainfall, in inches, for each season at Sweet-water Dam.	Run-off, in gallons.
1887–88	.....	2 302 581 600	1899–00	5.54	0
1888–89	13.53	8 250 155 100	1900–01	7.03	270 507 600
1889–90	13.53	6 707 804 400	1901–02	4.86	0
1890–91	12.65	7 045 938 900	1902–03	5.72	0
1891–92	9.88	2 024 886 600	1903–04	6.39	0
1892–93	11.62	5 312 142 000	1904–05	15.55	4 495 392 000
1893–94	6.20	437 124 600	1905–06	15.52	11 434 500 000
1894–95	16.19	23 983 700 400	1906–07	12.88	9 801 000 000
1895–96	7.29	431 244 000	1907–08	10.50	1 233 977 700
1896–97	10.97	2 251 289 700	1908–09	11.76	3 910 212 000
1897–98	7.05	1 306 800	1909–10	10.87	2 859 342 500
1898–99	5.05	80 041 500	1910–11	10.03	1 095 836 900
Total catchment in 7 years, 1897-1898 to 1903-1904......................					351 855 900

History of the Works.—The Lower Otay Dam (Fig. 2) was commenced in 1887, and a masonry base, 62 ft. wide, 28 ft. high, and extending 160 ft. across the valley of the Otay River, was built with the intention of making it a foundation for a masonry dam. The expense of the contemplated structure alarmed the owners, and they changed the type of construction to that of a rock fill dam with a central diaphragm of ¼-in. steel plate, protected on each side by 1 ft. of concrete. This dam was the first of the kind ever constructed, and has been described many times in technical publications. Operations on this new scheme were commenced in October, 1894, and finished on August 18th, 1897, when the dam was completed to the 130-ft. level above the old stream bed. The top of the dam is 615 ft. long and 15 ft. wide. The rock, taken from quarries adjoining, was

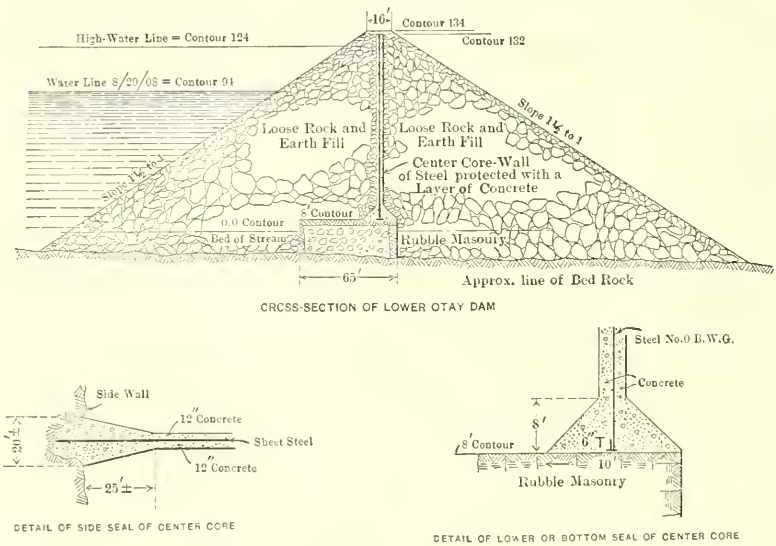


Fig. 2.

dumped on both sides of the core by Lidgerwood cableways, care being taken not to disturb the core when placing the rock adjacent to it. The slopes of the dumped rock are a little steeper than 1½ horizontal to 1 vertical. The dam has never been subjected to a full head of water, which would be 124 ft., or at the level of the spillway. In June, 1909, however, there was a depth of 109 ft. 6 in., which was lowered in December to 102 ft. Owing to the additional supply of water conveyed by the Dulzura Conduit, the lake level rose in May, 1910, to 119 ft. 6 in., and at present is about 110 ft. The high level of the water caused the saturation, for the first time, of the soft materials which were mixed with the rock from the quarry, and the result was a slight settlement of the fill on the up-stream side of the core—a condition naturally to be expected, as the fill is not composed entirely of clean rock. Additional materials have been hauled, however, to keep the top of the dam level.

Through this dam there has always been a slight leakage, and this fluctuates with the varying head in the reservoirs, as the formation is in a seamy porphyry, but it has not yet attained any serious proportions. This is one of the strongest objections to structures of this type, as it is not possible to determine the exact location of any leakage through the dam without endangering it by prospecting in the mass near the thin core.

As the area of the water-shed of Otay Creek above the dam is only about 100 sq. miles, with an average altitude of only 1 600 ft., the precipitation is light, and, except in occasional years, the run-off is only sufficient to fill the reservoir partly. To supplement and meet the requirements of the growing city of San Diego, the Company has built the Dulzura Conduit to divert the water from the Cottonwood. This drains an area of about 250 sq. miles of mountain water-shed (Fig. 1), which lies entirely in a granite formation. The Dulzura Conduit starts about 2 miles above the site of the Barrett Dam, and passes through it by a tunnel at an elevation of 60 ft. above the stream bed, or about 1 506 ft. above sea level, and is supported along the southerly slope of Lyon's Peak to Dulzura Divide, through which it passes through a tunnel, 970 ft. long, discharging into Dulzura Creek, which flows into the Lower Otay Lake.

This conduit is 13.38 miles long. It was commenced in August, 1907, and was completed on January 3d, 1909, under the writer's direction. It was built with a slope of 4 in 5 000, and consists of three types of construction, concrete-lined aqueduct, granite tunnels. and flumes. It has a carrying capacity, checked by weir measurements at its western terminus, of 44 000 000 gal. per 24 hours. Through the solid granite there are seventeen tunnels, 6 ft. wide and 7 ft. high, ranging in length from 40 to 2 060 ft., their total length aggregating 9 219 ft. The concrete-lined portion is 56 957 ft. long, lined on the sides with concrete from 4 to 6 in. thick. The bottom is not lined, as the materials found are either hard granite or impervious hardpan. The average width of this section is 5 ft., and its depth is 4 ft. 2 in. The side slopes are 3 in. horizontal to 1 ft. vertical. The length of the flume is 4 490 ft., and this is contracted to the shortest possible limit, owing to the fact that it is subject to destruction by fires, which in a dry summer and fall burn across the brush-covered country through which the conduit passes. The entire loss from seepage and evaporation does not exceed 3% of the capacity of the conduit.

Barrett Dam.—The site of the Barrett Dam is about 1½ miles below the junction of Pine Creek and Cottonwood Creek, and occupies a mountain gorge, with solid granite walls on each side. There is a drainage basin of 250 sq. miles above this dam site, and the Dulzura Conduit now passes through it. Extensive exploration and engineering studies have been made, and are still being prosecuted, to ascertain the best type of dam to build at this point. With a structure 150 ft. high, about 15 000 000 000 gal. of water can be impounded. The Dulzura Conduit has been constructed so that it passes through this dam site in a tunnel cut out of the solid granite, through which the water from the permanent outlet will be eventually passed. A wagon road, 12 miles long, has been cut out of the solid mountain side from the Barrett Dam to Dulzura, paralleling the conduit, so that cement and other materials for its construction can be handled economically.

Cottonwood Water-Shed.—The tributary water-shed of 250 sq. miles ranges in altitude from 1 500 to 6 000 ft., and probably averages 3 600 ft. The precipitation on this water-shed may ordinarily be expected to be from 10 to 20 in. greater than that of San Diego, due to altitude; and in some years it may be from 30 to 35 in. greater. The mean precipitation for San Diego for forty years, from 1850 to 1890, was 9.86 in., ranging from 3.02 in. in 1863, to 27.59 in. in 1884. To fill the reservoir to the 150-ft. contour will require 47 970 acre-ft., or 2 084 000 000 cu. ft., which would be supplied by an average rainfall of 3.6 in. from the water-shed. Under favorable conditions, this depth of run-off would be expected from an annual rainfall of 24 in., and, at times, may be the product of only 15 in. precipitation, depending largely on the distribution of the storms, and the frequency with which they succeed each other.

Morena Rock Fill Dam.—The other great dam of the Southern California Mountain Water Company, which this paper is intended

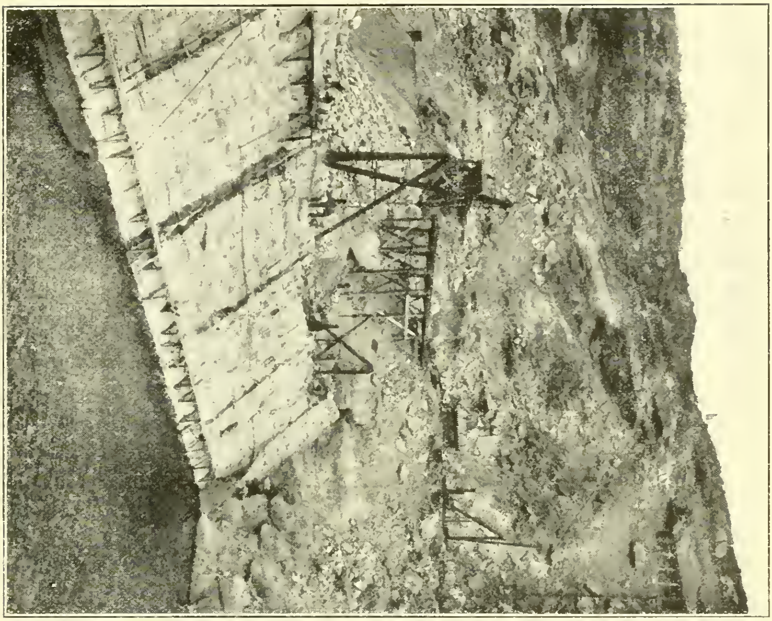


Fig. 3.—Morena Rock Fill Dam, Looking Northeast.

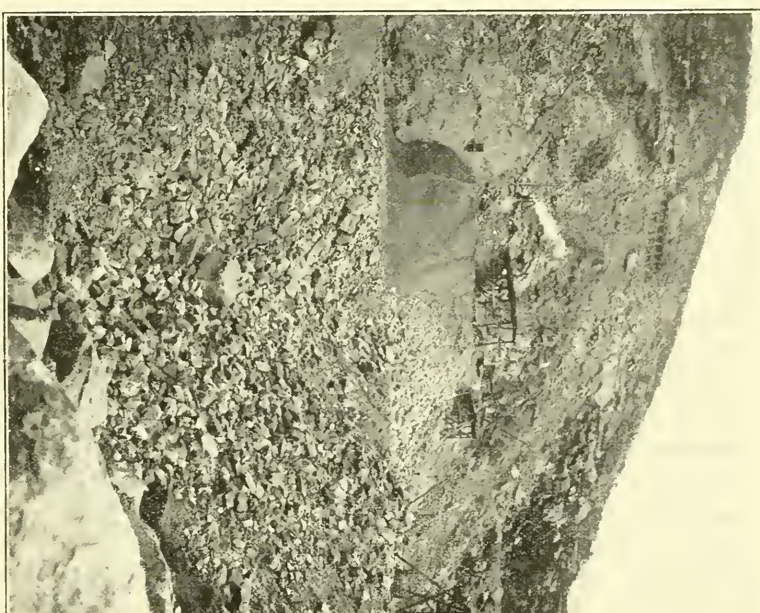


Fig. 4.—Morena Rock Fill Dam, Showing Loose Rock on Lower Slope, and Quarry Refuse Wasted Outside the Dam Section.

to describe, is 10 miles east of the Barrett Dam site, and on one of the streams which unite just above Barrett at an altitude of 3 100 ft. above sea level. It is 35 miles in an air line southeast from San Diego, and about 7 miles north of the Mexican boundary line. The dam is a rock fill structure (Figs. 5 and 6) facing a narrow cañon and lying between massive granite cliffs which tower 100 ft. high on the brink of a precipitous fall or cataract where the Cottonwood stream takes a plunge of 1 000 ft. within a distance of 2 miles. This dam forms one of a series of five to be constructed by the Company to impound water for the municipalities of San Diego and Coronado, and, ultimately, when an adequate reserve supply of water for domestic consumption is stored in the different reservoirs, to furnish water for the irrigation of extensive areas of frostless mesa land.

The Morena cañon is filled throughout with enormous boulders, and, at the site of the dam, a narrow fissure eroded by the stream was found to be 112 ft. below the stream bed at that point. A wall of rubble concrete, 36 ft. thick at the bottom and 12 ft. thick 30 ft. above the stream bed, was the first construction work done in 1896, when the project of building this dam was launched. At that time it was intended to build the water face on a slope of 1⅓ to 1, and the down-stream slope 1½ to 1; the top width was to be 20 ft. It was also intended to pave the water slope of the rock fill with an asphalt concrete, which was thought to be durable and impervious. Work was suspended in April, 1898, however, after the toe wall, up to the 30-ft. contour, had been completed and about 120 000 cu. yd. of rock fill had been placed, out of a total of 306 000 cu. yd. required to complete the dam to the prescribed cross-section. On resuming construction, on May 6th, 1909, it was decided to change the upper slope, from the top of the completed toe wall up to the 120-ft. contour, to 9 horizontal to 10 vertical, and make it, from the 120-ft. contour to the top of the dam—150 ft.—½ horizontal to 1 vertical. It was decided to alter the character of the work by placing large masonry blocks of roughly dressed granite, from 6 to 10 tons in weight, selected from the rock piles, on the up-stream face of the dam, and have them well bedded and set in cement mortar composed of 1 part cement to 2½ parts sand, and behind this masonry skin, about 7 ft. thick, to place all the stone, by hand and derrick, for a width of

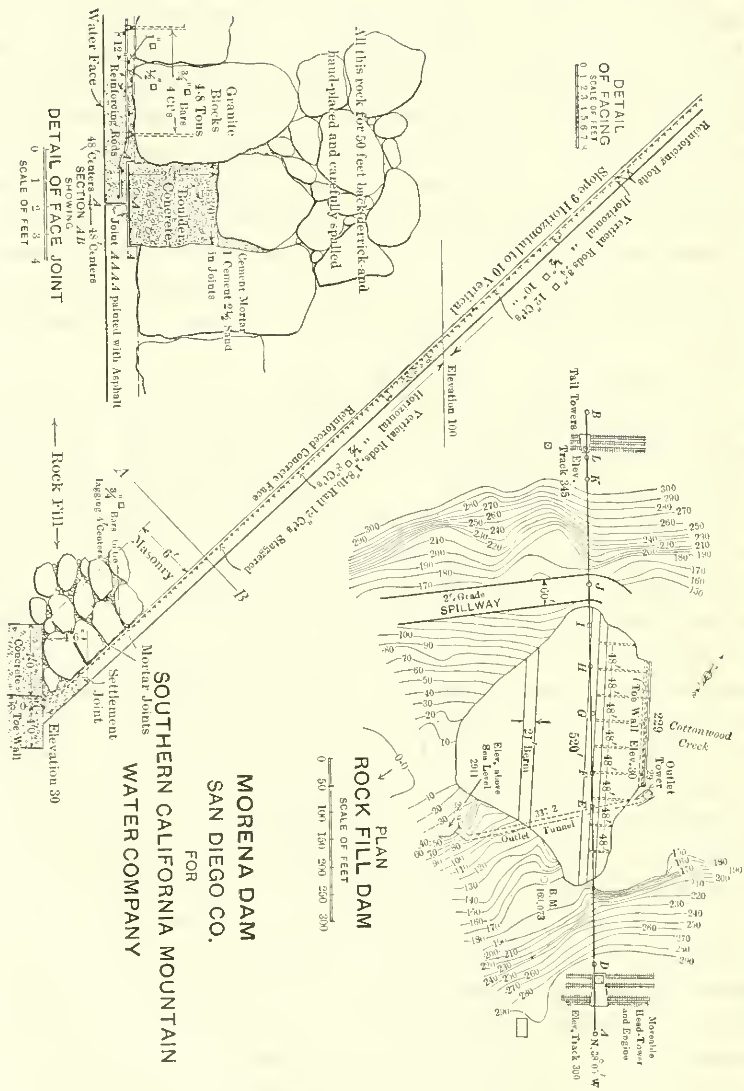

MORENA DAM
SAN DIEGO CO.
FOR
SOUTHERN CALIFORNIA MOUNTAIN
WATER COMPANY
PLAN
ROCK FILL DAM
SCALE OF FEET
0 50 100 150 200 250 300
DETAIL
OF FACING
SCALE OF FEET
DETAIL OF FACE JOINT
SHOWING
SECTION AB
SCALE OF FEET
0 1 2 3 4
Slope 9 Horizontal to 10 Vertical
Reinforced Concrete Face
Reinforcing Rods
Elevation 100
Elevation 30
Rock Fill
Water Face
Granite Blocks
4-8 Tons
Boulder Concrete
Cement Mortar
1 Cement 2½ Sand
in Joints
All this rock for 50 feet back derrick-and
hand-placed and carefully spalled
Joint AAAA painted with Asphalt
Masonry
Mortar Joints
Settlement Joint
Toe Wall
SPILLWAY
Outlet Tunnel
Outlet Tower
Cottonwood Creek
Berm
Tail Towers
Movable Head-Tower and Engine
Elev. Track 345
Elev. Track 300

Fig. 5.

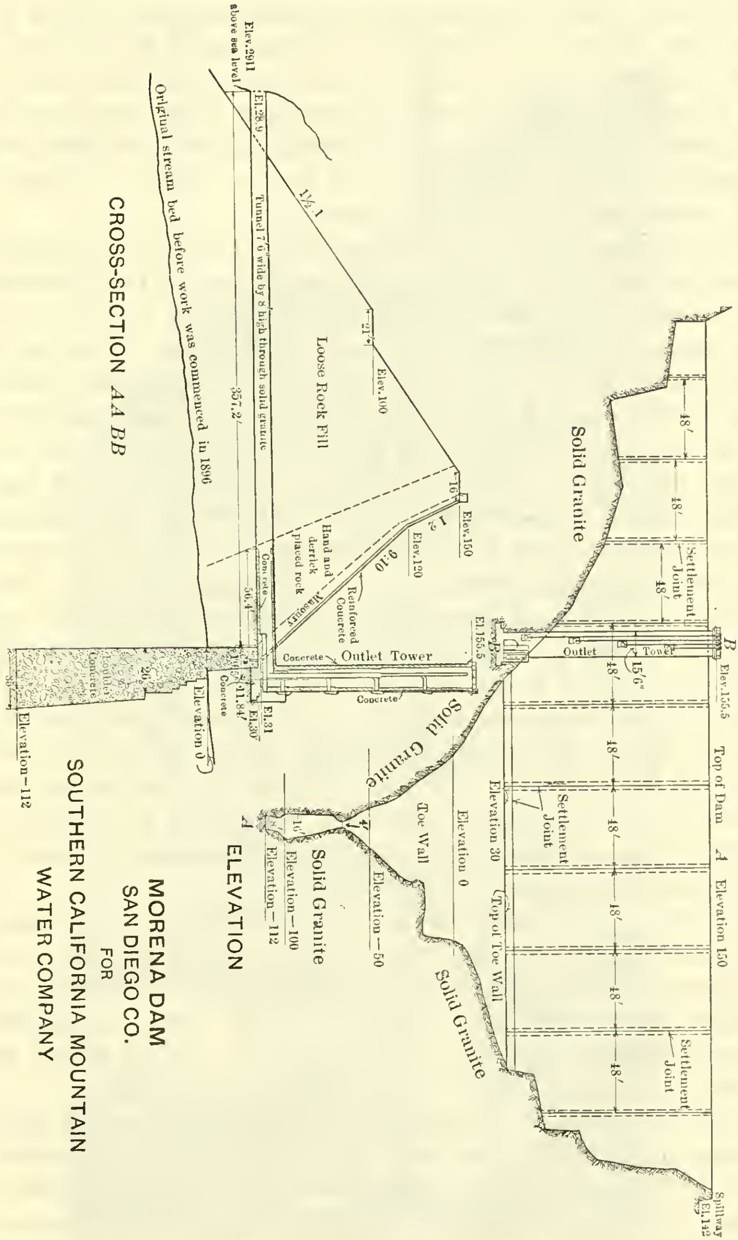

MORENA DAM
SAN DIEGO CO.
FOR
SOUTHERN CALIFORNIA MOUNTAIN
WATER COMPANY
CROSS-SECTION AA BB
ELEVATION
Loose Rock Fill
Hand and derrick placed rock
Reinforced Concrete
Masonry
Outlet Tower
Tunnel 7′6″ wide by 8′ high through solid granite
Original stream bed before work was commenced in 1896
Elev. 2911 above sea level
Solid Granite
Settlement Joint
Top of Dam
Elevation 150
Top of Toe Wall
Toe Wall
Spillway El. 142
Elev. 155.5

FIG. 6.

50 ft. back from the face, in order to provide consistent support for the skin. The top of the dam is 16 ft. wide, and is crowned with a 3-ft. concrete coping to provide for wave wash. The back slope is $1\frac{1}{2}$ horizontal to 1 vertical, with a berm of 21 ft. at the 100-ft. contour to provide for future extensions in raising the dam. This berm was practically created by the alteration of the face slopes, which originally contemplated a flatter water slope.

In the original work, done 12 years ago without proper continuous technical supervision, there was a little recklessness in the disposition of the materials in the dam, and it was necessary, when the work was resumed in 1909, after a thorough examination, to tear out a large part of the old fill behind the toe wall, remove all loam or other objectionable material, and rebuild with a clean, well-placed rock fill.

At one time, after seeing the nature of the old construction, the writer was almost committed to the idea of letting the old mass stand and, as the additional rock was being placed, hydraulicking a granite surface soil from the adjoining hills into the interstices as the dam was being raised. After a thorough discussion of the matter, however, with many brother engineers, and especially with George L. Dillman, M. Am. Soc. C. E., who seemed to have a clearer conception than any one interviewed as to the requirements of a rock fill dam, the present method of procedure was decided on; that is, to construct a water-tight skin for the face of the dam and keep the rock fill absolutely clear of any soil or silt, and thus provide free drainage for any possible leaks which might occur in the future, without impairing the integrity of the mass by washing out sand or other solvent materials.

To obviate further any serious cracking of the skin which might be caused by settlement, grooves, 3 ft. square, and 48 ft. from center to center, were placed in the masonry, and into these concrete has been poured and brought to an absolutely smooth surface, the general average plane of which projects 3 in. beyond the general face of the masonry.

In the future it is proposed to place reinforced concrete slabs, averaging 1 ft. in thickness, on the face of the masonry and attached to it by $\frac{3}{4}$-in. iron rods, 4 ft. from center to center. These slabs will be joined with $\frac{3}{4}$-in. water-tight settlement joints, poured with a sand asphalt along the center line of, and resting on, the concrete grooves

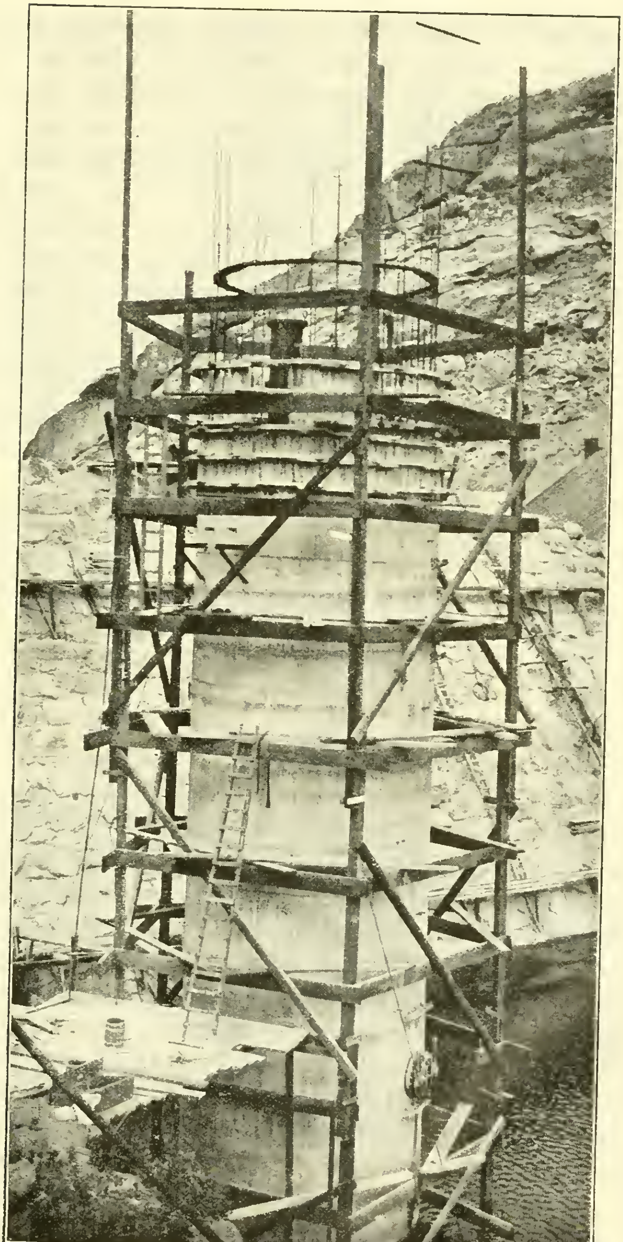


FIG. 7.—REINFORCED CONCRETE OUTLET TOWER, MORENA RESERVOIR.

previously mentioned. A slot, 1 ft. wide and 5 ft. deep, was left in the old toe wall, Fig. 5, into which the reinforced concrete facing has already been nested and constructed up to the 42-ft. contour, or 12 ft. above the old toe wall. The zero of the dam levels, as shown by the drawings, is the surface of the original stream bed before work was begun.

This toe wall had a leakage of 57 800 gal. per day in 1898, after it was built, with a head of water up to the 30-ft. contour. It will be noticed from Table 4 that the present leakage, under a head of 65 ft., is only 33 604 gal. per day. This is now measured at a bed-rock outcrop in the cañon below the dam and about 1 000 ft. from it.

TABLE 4.—LEAKAGE OF MORENA RESERVOIR.

Date.	Depth of water, in feet.	Leakage, in gallons per 24 hours.
January 13	42	29 727
March 6	48	97 226
March 8	50.5	97 226
March 11	53	120 596
March 14	55	104 152
March 19	57.15	104 152
March 25	58.5	104 152
April 5	60.37	70 448
April 10	61.27	70 448
April 16	61.97	70 448
April 19	62.3	51 053
April 25	63.0	46 529
May 1	63.5	51 053
May 7	63.93	51 053
May 12	64.25	46 529
May 14	64.37	51 053
May 22	64.78	37 481
June 5	65.33	33 604

As the freight rates from San Diego to Morena—a distance of about 60 miles by wagon road, with maximum grades of 16% to be overcome—run up to about 1 cent per lb., it has been imperative to reduce to a minimum the quantities necessary to make a first-class job, on account of the excessive cost of hauling at the dam site.

The cañon walls at the dam site are all of clean, hard granite, singularly free from fissures and seams, and the width between them is only 80 ft. at the stream bed and 520 ft. at the 150-ft. contour—the level of the top of the present completed dam. It has been necessary to add about 186 000 cu. yd. of rock to the original 120 cu. yd., in order to complete the dam to the prescribed cross-section. Construction operations were prosecuted with two Lidgerwood cableways, which were operated from towers about 300 ft. above the stream bed. The fixed cable was $2\frac{1}{2}$ in. in diameter and 1 350 ft. long, covering the lower slope of the dam. The other cable was mounted on movable trucks, Fig. 5, which had a movement of 170 ft. on tracks at right angles to the axis of the dam, and was able to cover the whole of its water slope; this cable was $2\frac{1}{4}$ in. in diameter and 1 100 ft. long. Each cable was able to handle readily loads up to 10 and 12 tons. All the large rock was chained, and was either picked up by the cable directly or delivered to it by feeding derricks, and transferred by the cable to the fill; the smaller rock was carried in 6 by 8-ft. skips, each having a capacity of 2 cu. yd. It was possible to move the track trolley cable into a new position in about 2 hours, which made a very convenient arrangement for moving the stone from the quarries directly into the dam work, where it was re-handled by derricks for the face masonry and back-filling.

Signaling Apparatus.—In order to control the rapid operation of cables and the exact delivery of rock at certain points of the dam, and to communicate directions to the operating engineers, who were unable to see the work, a new system of signaling was devised. The system of bells used in the old days was abandoned, and an annunciator consisting of a box having ten compartments, each 8 by 8 in. deep, was placed within view of the engineer. The front of each compartment was closed by a pane of frosted glass and on these the following signals were painted: "HOIST", "LOWER", "GO OUT", "COME IN", "FAST", "SLOW", "STOP", and three spare spaces were left for special signals. At the back of each compartment were mounted two Edison keyless wall sockets with 16-c-p., 110-volt lamps. The lamps were wired with a common return wire and an individual wire for the other terminal of each lamp, making 11 wires in all. These wires were of No. 14 copper, covered and cabled, and the outside was protected by jute braid. Each flexible cable was 650 ft. long, and could readily be moved to any favorable position on the south or operating end of the dam. At the signaling end of the cable, ten switches were mounted and normally held open by a spring requiring the pressure of the operator's fingers to close it. The switches were mounted on an insulated base in such a way that the leads were brought into them without coming into contact with the wooden framework. A $1\frac{1}{4}$-kw., 125-volt, direct-current, compound-wound generator was used, operating at a speed of 1 650 rev. per min. The generator

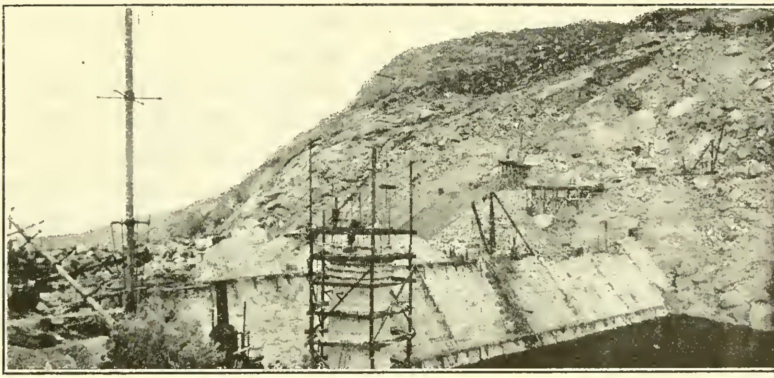


FIG. 8.—ROCK FILL DAM, MORENA RESERVOIR.

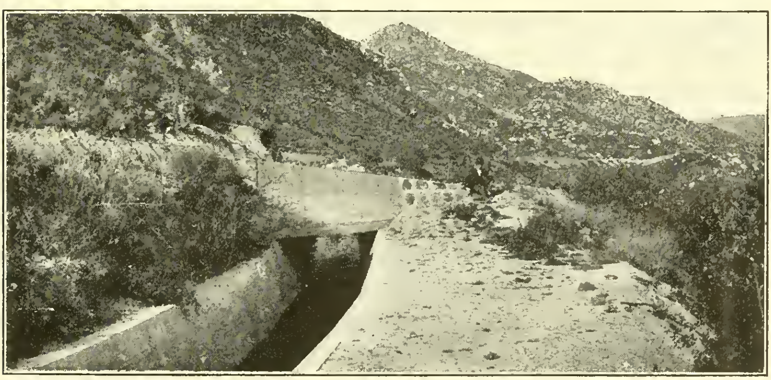


FIG. 9.—DULZURA CONDUIT, NEAR ITS WESTERN END.

was driven by one C. H. Dutton, 5-h.p., vertical, steam engine, at a speed of 300 rev. per min. The engine was supplied with steam tapped from one of the boilers of the big Lidgerwood engines. By this system, a man, with the switch-signal board, moved around the dam to the most effective points for observing the control and placing of rock as the work progressed, without interfering with guy wires and absolutely preventing accidents, for, in 2½ years of operation, not a man was injured because of any confused signals.

Blasting Rock.—In the construction work of 1896 and 1898, large blasts of powder were used indiscriminately in the adjoining hills to break up the solid granite for handling. Powder was used somewhat generously, as at that time the Company obtained it at the very low contract price of 5 cents per lb. As the price of Judson powder was raised to 8 cents in 1909, and other high-grade powders proportionately, a very careful study of the whole formation surrounding the dam was made before deciding to break rock for the new work. The whole formation in the nearby hills, from which the material for the dam was taken, is a solid granite, and as the cableways were fortunately placed at such a high elevation, 300 ft. above the dam site, a splendid opportunity presented itself for making a mass shot with a large charge of powder without endangering in any way the formation supporting the dam.

A careful study of the rock formation suggested the following means of attack: North of the spillway, Fig. 10, an open cut, 100 ft. long, 12 ft. wide, and about 40 ft. deep, was made at the toe of an old quarry. Parallel to this, and 100 ft. distant, a 4 by 5-ft. tunnel drift, 115 ft. long, was driven into the solid granite, and 70 ft. back from the portal a small chamber, *Y*, depressed below the level of the bottom of the drift, was excavated and made large enough for a 3¼-ton powder charge. At the extreme end of the tunnel drift, a chamber, *X*, 8 by 8 by 14 ft., depressed below the floor and staggered to the left from the general direction of the tunnel, was excavated, and in it a 16.225-ton powder charge was placed. The grade of the tunnel drift was also made 10 ft. above the level of the open cut, conforming to the natural dip of the formation, which was about 10° to the southwest. This layout was designed to throw the broken rock into the area above the open cut, and it was accomplished successfully by the explosion, which took place on August 30th, at 4.20 P. M.

In the large chamber there were 571 50-lb. boxes of 7% and 9% "Champion" powder, 38 boxes of No. 2, 40% dynamite, weighing 2 000 lb., and 40 boxes of No. 1, 60% dynamite, weighing 1 900 lb., in all 32 450 lb., or 16.225 tons. In the smaller chamber there were 100 boxes of 7% "Champion" powder, weighing 5 000 lb., and 30 boxes of No. 2, 40% dynamite, weighing 1 500 lb., in all 6 500 lb., or 3.25 tons.

Cost of Shot.—The detailed costs were as follows:

Tunnel drift and loading	$2 478.15
Open cut	3 499.80
Powder in chambers	3 116.00
Total	$9 093.95

From this sum must be deducted 1 400 cu. yd. of solid rock excavated from the cut and the tunnel, worth $1 400, which left a net cost of $7 693.95, or 4.274 cents per ton for 180 000 tons.

Description of Loading.—In large blasts where powder forms the bulk of the charge, it is preferable to sink pits below the floor level of tunnels or drifts to hold the charges, on account of the greater facility with which a loose powder is loaded into a pit, since greater compactness of the charge is obtained by the men in the pit constantly treading the powder while placing the charge in position. In addition, the sides of a pit offer greater resistance than it is possible to obtain where the powder is loaded directly into the end of the tunnel or drift. This greater resistance offered by the pit is of the utmost importance because it determines the quantity as well as the quality of the explosive necessary, and maximum results will be obtained only where the powder has been confined until the maximum pressure has been attained. The seemingly large primers of dynamite used in this blast, *viz.,* 2 000 lb. of 60% and 1 900 lb. of 40% in the tunnel pit, *X,* and 1 500 lb. of 40% in the drift pit, *Y,* were used first to obtain instantaneous maximum pressures, this being dependent on the rate or speed of detonation of the dynamite, and secondly, for their percussive action. Of course, the rate of detonation and percussive action of the dynamite greatly outstrip the maximum pressure developed by the powder.

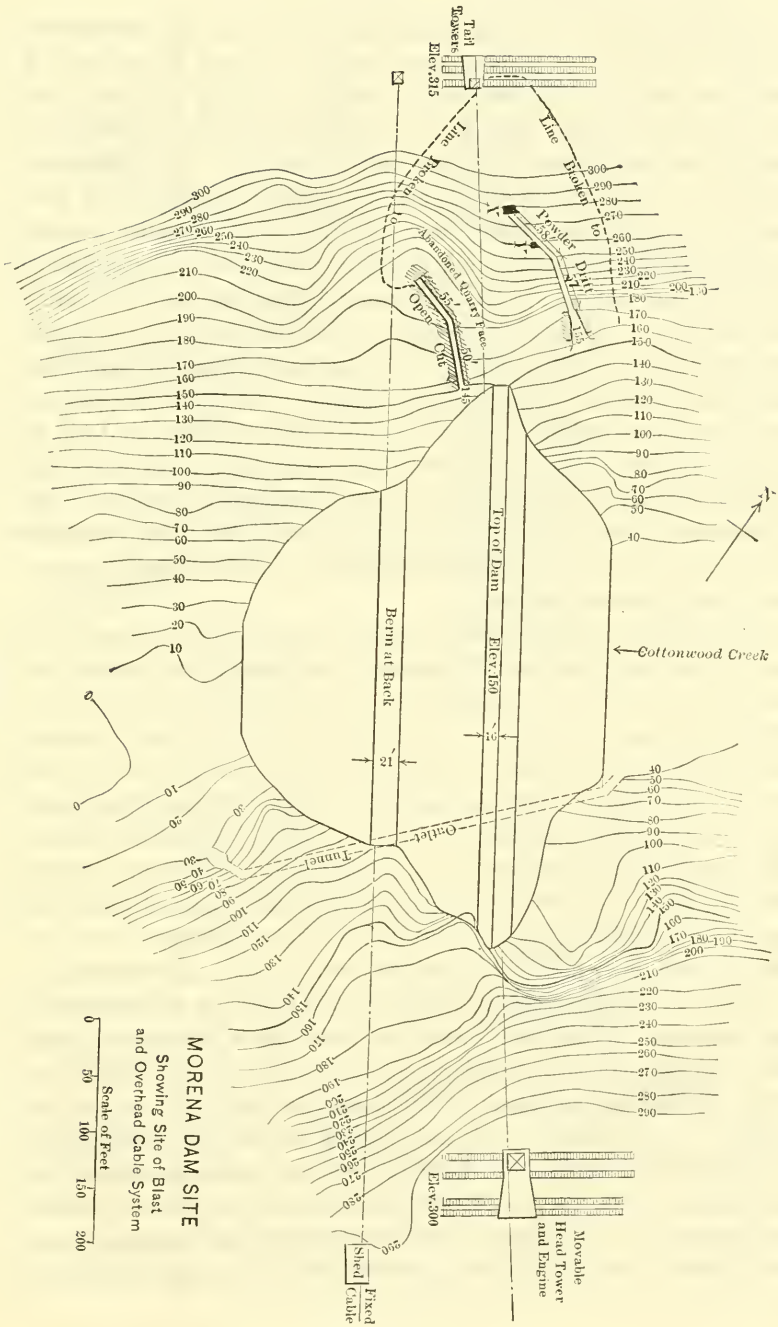
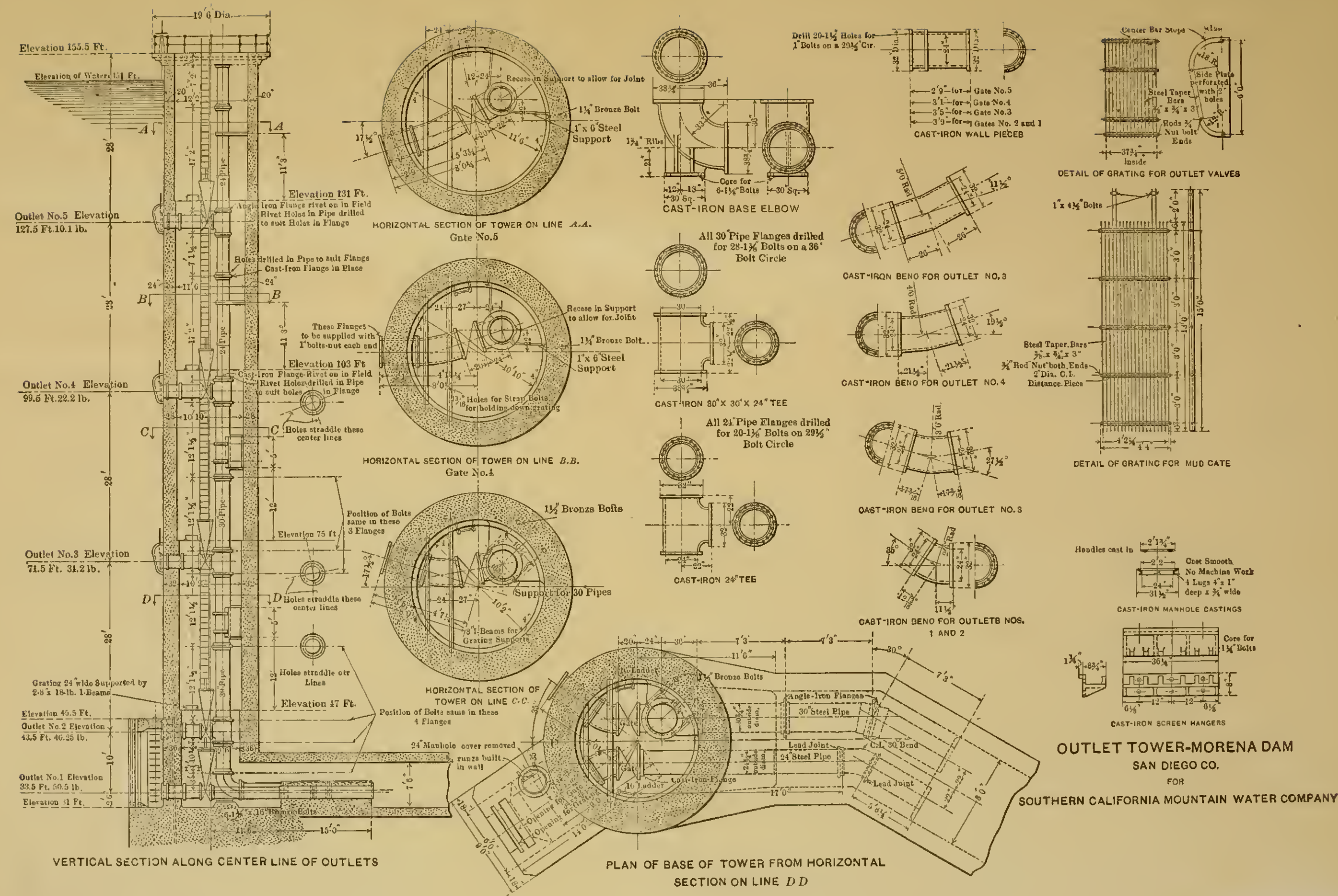

MORENA DAM SITE
Showing Site of Blast
and Overhead Cable System
Scale of Feet
0 50 100 150 200
Tail Towers
Elev. 315
Line Broken
Line Broken to Powder Drift
Abandoned Quarry Face
Open Cut
Top of Dam
Elev. 150
Berm at Back
Outlet Tunnel
Cottonwood Creek
Elev. 300
Movable Head Tower and Engine
Shed
Fixed Cable

FIG. 10.

The first step in charging the tunnel pit, *X*, was the introduction of about 10 000 lb. of powder, then 2 000 lb. of 60% dynamite in a semicircle was stacked, two boxes high, in contact with the west wall of the pit, then 1 900 lb. of 40% dynamite was stacked in a similar manner against the east wall of the pit. The percussive action of these primers was to start a break or shear in the rock about 2 ft. below the tunnel floor and in a plane parallel with it, allowing the more slowly forming gases of the powder to expand horizontally, thus giving a greater horizontal purchase and preventing the formation of a crater or blow-out. The remainder of the charge of powder was placed on top of the dynamite primers, all spaces being filled completely with the loose powder. The drift pit, *Y*, was loaded first with 2 500 lb. of loose powder. Then a primer of 1 500 lb. of 40% dynamite was stacked, two boxes high, along the north wall of the pit on top of the powder, with the intention of starting a break or horizontal shear in the direction of the tunnel pit. The remainder of the charge, 2 500 lb. of powder, was then placed in the pit and on top of the dynamite primer.

The wiring consisted of 20-ft. double-strength electric fuses, from each of the three primers of dynamite to the main tunnel, where they were connected in series to the battery at the portal. The wire connections between the primers were: a No. 16, B. & S. gauge, insulated copper wire from the battery to the pit *X* 60% primer, thence to the pit *X* 40% primer; a No. 24, insulated copper wire from this latter to the pit *Y* primer, and thence a No. 16 wire back to the battery. The wires were covered with 6 in. of fine earth, and were protected further at the drift portal by being enclosed in a grooved board laid across the portal and covered with earth. The resistance of the circuit at the tunnel portal was 8 ohms, by a Dupont galvanometer.

The first tamping used was fine earth, well tamped on the powder, and completely filling both tunnel and drift for a distance of 10 ft. back from the charges. Bulkheads of rock were placed at intervals of 10 ft. throughout the length of the tunnel in order to tamp its roof solidly. At the drift portal and locking against the opposite wall of the tunnel, a bulkhead of heavy green oak timbers was placed and well tamped. The earth and rock tamping was then carried forward to within 20 ft. of the tunnel portal. If the ground had been conglomerate instead of exceedingly hard granite, the total weight of dynamite



OUTLET TOWER-MORENA DAM
SAN DIEGO CO.
FOR
SOUTHERN CALIFORNIA MOUNTAIN WATER COMPANY

primers used would have been 500 lb. instead of 5 400 lb., for in conglomerate the percussive action of dynamite would be waste energy, inasmuch as the action required would be heaving, and the sole use of dynamite in the charge would be to control the rate of detonation of the nitro-glycerine content of the "Champion" powder. In the hard granite at Morena a rending as well as a heaving action was required, in order to produce adequate results, hence the use of a comparatively large quantity of dynamite.

During the loading and handling of the large quantities of powder, it was customary to require that all the men thus engaged surrender matches and smoking materials, and that the men in the pit remove their shoes and wrap their feet in sacking. The circuit was tested frequently at the tunnel portal during tamping in order that possible breaks be detected and repaired before burying the wires too deeply. The battery was tested up to full strength before using. Light was furnished by candle lanterns securely wired to pegs driven into bore holes in the sides of the tunnel and drifts.

The loading of the charge was surpervised by Mr. J. S. Molony, Resident Engineer, and Messrs. A. H. Crane and H. F. Smith, Agents for the Dupont Powder Company.

On August 30th, 1909, the blast was exploded electrically, and the result was most satisfactory, as it displaced about 180 000 tons of rock at the moderate price stated. The greater part of the whole mass of granite was broken into blocks of from 50 lb. to ¼ ton in weight. The fact that about 75% of the pieces weighed less than 10 tons enabled the cables to do very effective work with little plugging.

Outlet Tower.—Through the solid bed-rock on the south side of the dam at the 30-ft. contour, a tunnel, 387 ft. long, 8 ft. wide, and 7½ ft. high, was drifted, through which the water from the reservoir is drawn off. This is done through a reinforced concrete tower, Plate II, 15 ft. 6 in. in external diameter, with walls varying in thickness from 3 ft. 0 in. to 20 in. At the top of this tower, at an elevation of 155½ ft., there is an operating deck of reinforced concrete from which the outer gates are regulated. These gates are of the Coffin Valve Company's sluice type with vertical stems controlled by guides let into the concrete. Around each gate there is a screen to keep trash and drift from entering the 24-in. circular, cast-iron pipes passing through the walls of the tower and connecting with a 30-in. vertical down-pipe which discharges into the tunnel. These outlets are 28 ft. apart vertically, so that the water may be drawn off under a light head from any of these levels. Between each opening and the down-pipe there is a curved, removable, bolted, flanged casting to enable the gates to be easily removed, if it is ever found necessary. Each is attached to Crane gates which are operated from platforms inside the tower at the different levels, and are used for emergency purposes only. The admission of water through the outer sluice-valves relieves the leaf-valves from any chattering effect developed by the spouting water which discharges into the down-pipe and flows freely through the tunnel. There is also an independent 24-in. cast-iron pipe which will be used for washing out the sediment which, in the future, may accumulate near the base of the outlet tower. This pipe passes through the foundation of the tower and is connected with the tunnel. The inner 75 ft. of the tunnel—in solid granite—is lined with concrete; it is connected with the base of the tower, and encloses the pipes which discharge at the floor of the tunnel, so that every precaution has been taken to obtain, at a moderate expense, as simple and safe an outlet as possible. From the outer end of the tunnel the water will be permitted to flow at present along the natural grade of Cottonwood Creek until it is picked up above the Barrett Dam site by the Dulzura Conduit.

The work has been done by day's labor under the supervision of Mr. J. S. Molony, acting as Resident Engineer and General Superintendent, with Messrs. R. Wueste and R. P. McIntosh as Assistants. About 95 men have been constantly employed since the blast explosion of August 30th, 1909, and the work has been peculiarly free from accidents, owing to the careful and conscientious supervision exercised by the men in charge.

The face masonry and back-filling have been expensive, the average cost of the whole work having been $2.50 per cu. yd., but the actual placing of the rock in the dam from the quarries has not cost more than $1 per cu. yd. The whole plant had to be rehabilitated on resuming work in 1909, which, of course, has added slightly to the unit costs, though labor conditions at present in that part of California have made work cost about 30% in excess of prices twelve years ago.

The writer has acted as Chief Engineer and Consulting Engineer of the project, and has not been restrained in any way, by the President of the Company, Mr. Spreckels, who practically owns the entire water system, from modifying the plan of operations or type of construction to suit conditions as the work progressed.

As San Diego is growing rapidly, having a population of 45 000 at the present time, every precaution is taken by the Southern California Mountain Water Company to conserve a sufficient supply of water to take care of the increased population and growth which the Panama Canal and other factors are bound to create.

DISCUSSION

Mr. Dillman.

George L. Dillman, M. Am. Soc. C. E. (by letter).—As Mr. O'Shaughnessy has given the writer some credit in the paper, it may not be amiss to enlarge on the great "Hydraulic Principle." This principle applies to practically all hydraulic construction. It has been persistently ignored by many alleged experts, always to the detriment, and often to the destruction, of the works. Where intelligently applied, the result has been safety and economy. Just why it has not been announced by the great writers on hydraulics and taught as a fundamental principle in engineering schools has never been apparent to the writer.

Briefly stated, the principle is this: Construct one impervious surface, and build the rest of the structure to support that surface. If this surface should not be water-tight, make it as nearly so as possible, in order that seepage or leakage will not be allowed to accumulate pressure against some other surface, or do other damage in getting away.

In the case of masonry or concrete dams, the particular part to make tight is the up-stream face. If this is tight and supported, the result will be a dam; otherwise, a failure. This support is: (1) Solid masonry or concrete, the result being the so-called uniform-sectioned type; or, (2) Buttresses of masonry or concrete, resulting in the multiple-arch dam—safer and more economical; or (3) A mass of loose rock, as in the present case, resulting in a stable structure.

In (1), special care must be taken that the up-stream face is the least pervious surface, because, making any other surface tighter would tend toward weakness. Carried to the limit, making the down-stream surface of ordinary types the most nearly water-tight would insure failure. In (2), the multiple-arch or buttressed-wall type, no special care need be taken. Seepage will find its way out through the arches. In (3), or the present case, the loose rock fill takes care of the seepage if only ordinary care is taken in clearing the foundation for it.

The particular method of supporting the impervious (or most nearly impervious) surface is generally a matter of dollars and cents. That the author has chosen the best one must be conceded, when the price of cement is considered. In other words, the loose rock wall undoubtedly costs less than any other support. To have placed any fine material in the loose rock would have been a mistake, not necessarily fatal, but an added expense, possibly resulting in failure. It costs money, does no good, and may be harmful; yet it has been done recently in California.

In the case of earth dams, with special cores of puddled clay, masonry, or steel, earth on the up-stream face acts in one of two ways.

Mr. Dillman.

It may be impervious, making the core unnecessary, or it may be merely a support to the impervious core. This earth generally needs rip-rapping or paving, in order to resist wave action. In the case of puddle, it is necessary to prevent the puddle from drying out. In the case of masonry or steel, it seems to be unnecessary, as masonry or concrete could be built on a batter which would obviate the need for support. As steel is not a permanent construction material, its life would depend on its coating, which itself might require some protection; but the down-stream material should be pervious. All possible seepage through the core should get away without eroding the material or producing pressure.

In the case of timber dams, the structure that stands is a face of planking connected with sheet-piling, as tight as the builder can make, supported by cribs, bents, loose rock, or something else, but always following the "Hydraulic Principle." Tightening it in more than one place is expensive and often fatal.

Other structures may well be mentioned here. A retaining wall should always have drains through it to prevent it from becoming a hydraulic structure. A steel tank or steel pipe should always be caulked on the inside, as the outside caulking only forms a small lip for tightness and soon rusts off or is forced open, whereas the inside caulking has the necessary support. In a reinforced concrete reservoir, the concrete forms the impervious surface, the steel its support. The location of the reinforcement and its initial tension are matters of moment. If the reinforcement is put in without initial tension, the concrete must give before the strength of the reinforcement is developed—this is the most common cause of leaky structures. Other cases of application cannot fail to occur to readers. The application of the principle cannot help but be efficient and economical.

This may not be a discussion of the paper in the sense usually intended. The author seems to have covered the subject completely. California has a great number of dams, and many extremes of types, and this seems to add another, for the writer knows of no rock fill dam of greater height. San Diego is to be congratulated on the assurance of a continued and increased water supply, Mr. O'Shaughnessy on having constructed a great dam, and the Society on having such a complete description of it.

Mr. Maddock.

George F. Maddock, Esq. (by letter).—In connection with the data in this paper, it may be of interest, to those who have to do with hydraulic development in semi-arid countries, to present a brief synopsis of a report made by Mr. O'Shaughnessy for the writer, on the run-off from 210 sq. miles of the water-shed of the San Luis Rey River, in the eastern part of San Diego County, California. In order to study these conditions, isohyetose lines, or lines of equal rainfall, were drawn on the United States topographical map of Southern Cali-

Mr. Maddock.

fornia. The data for locating these lines were secured from the United States Weather Reports and from private records. The rainfall stations covered a wide area, and varied in elevation from sea level to 5 300 ft. As will be seen from Table 5, the observations covered many years. These isohyetose lines divided the water-shed into precipitation zones, and the rainfall on each zone was estimated by assuming the average between each boundary. The run-off was computed from percentages obtained from the Cottonwood observations.

TABLE 5.—Rainfall Stations, San Diego County, California, and Vicinity.

Name of station.	Elevation above sea, in feet.	Length of observation period, in years.	Average annual rainfall, in inches.
Elsinore	1 234	12	13.64
Fall Brook	700	27	17.14
Valley Center	1 365	26	20.03
Escondido	657	14	15.15
Poway	460	29	13.79
El Cajon	482	10	12.24
San Diego	Sea	53	9.62
Sweetwater Dam	238	20	9.52
Jamul	900	6	13.00
Barrett Dam	1 600	5	19.07
Campo	2 189	31	19.98
Morena Dam	3 300	5	24.15
Buckman Springs	3 500	2	19.90
Noble's Mine	4 200	3	24.5
Cuyamaca Reservoir	4 677	21	38.84
Julian	4 250	28	26.36
Santa Ysabel	2 983	10	24.17
Mesa Grande	3 300	5	30.70
Nellie	5 300	7	44.26
Warner's Springs	3 165	4	16.08
Salton Sea	Sea	30	3

Tables 5, 6, and 7 give the precipitation records, the run-off observations on the Cottonwood water-shed, and the estimated run-off for the 210 sq. miles of the San Luis Rey water-shed (Warner's Ranch).

TABLE 6.—Run-Off from Cottonwood Water-Shed above Barrett.
Area, 250 sq. miles.
Elevation, 1 506 to 6 000 ft.

Year.	Mean rainfall, in inches.	Run-off, in inches.	Run-off, in acre-feet.	Percentage of run-off to rainfail.
1906	32.33	3.7	59 870	11.5%
1907	15.68	2.12	34 000	13.4%
1908	18.69	0.81	12 970	4.3%
1909	28.76	1.82	29 190	6.3%
1910	12.61	1.05	16 870	8.3%
Total for 5 years	....	......	152 900	43.8%
Mean	....	.....	30 580	8.76%

TABLE 7.—ANNUAL RUN-OFF FROM WARNER'S WATER-SHED.

Mr. Maddock.

Rainfall.	Area.		Average depth of rainfall, in inches.	Percentage of run-off.	Run-off, in inches.	Seasonal run-off, in acre-feet.
	Acres.	Square miles.				
Between 45 and 40 in........	6 080	9.5	42.5	18.8	8	4 054
" 40 " 30 "	9 216	14.4	35	12.8	4.5	3 456
" 30 " 25 "	11 520	18	27.5	8.0	2.2	2 112
" 25 " 20 "	16 128	25.2	22.5	7.5	1.7	2 293
" 20 " 15 "	55 680	87	17.5	7.0	1.2	5 568
" 15 " 10 "	35 776	55.9	12.5	6.5	0.81	2 415
Totals....................	....	210.0	....	....	....	19 848

The foregoing analysis of this water-shed, when proper storage was considered, indicated that only 20 cu. ft. per sec. were available for power during the long periods of drought which occur in this locality. This is slightly less than 0.1 cu. ft. per sec. per sq. mile of water-shed, and agrees remarkably well with the Government observations of this stream at Pala.

Mr. Galloway.

J. D. GALLOWAY, M. AM. SOC. C. E. (by letter).—This dam is a notable addition to those of the type which have been built in Western America, and is a credit to Mr. O'Shaughnessy, the designer and builder.

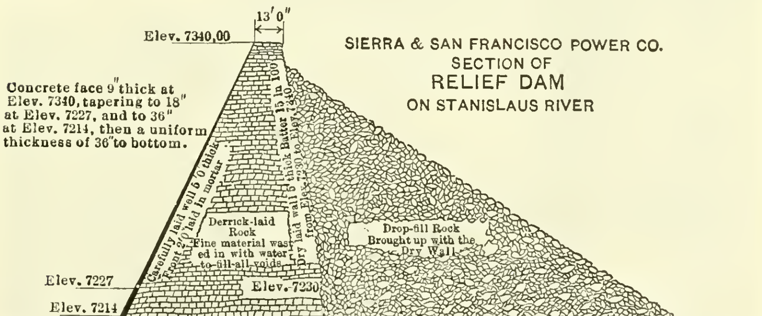
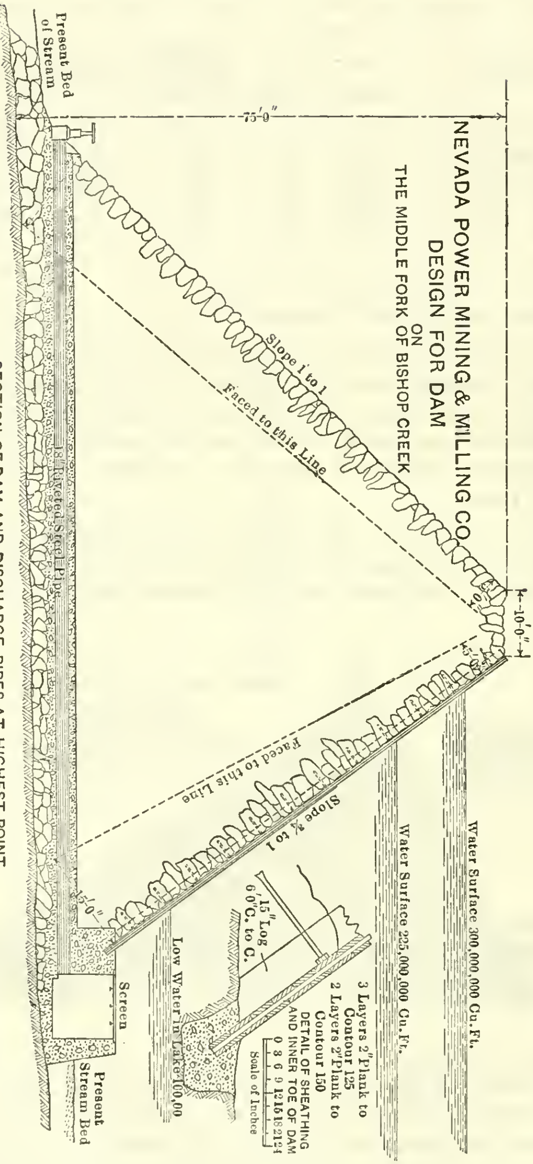


FIG. 11.

The Morena Dam differs in one respect from other rock fill dams in the method of constructing the face or water-tight skin. Mr. O'Shaughnessy has made the face of large stones set in cement mortar, and from his description it would seem that it is the intention to allow water pressure against this face before the concrete slabs, 1 ft. thick, are added. It would be interesting to know the head of

Mr. Galloway.



SECTION OF DAM AND DISCHARGE PIPES AT HIGHEST POINT

Fig. 12.

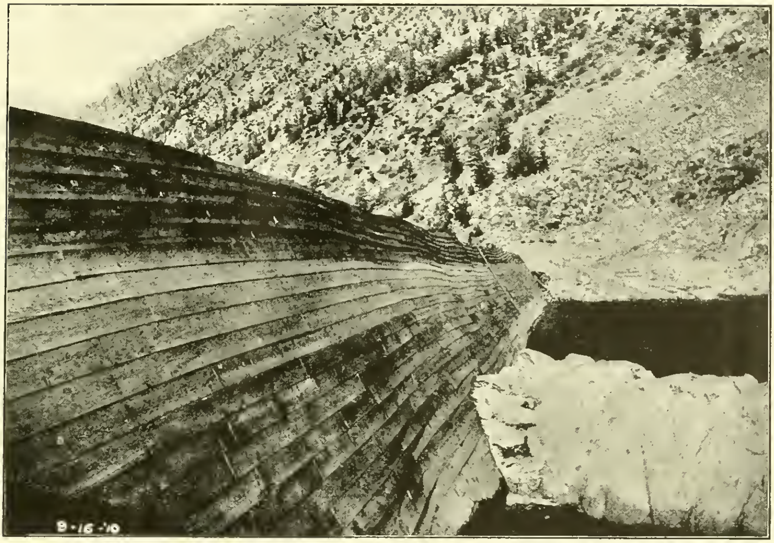


Fig. 13.—Timber Sheathing of Rock Fill Dam on Middle Fork of Bishop Creek, California, for the Nevada-California Power Company.

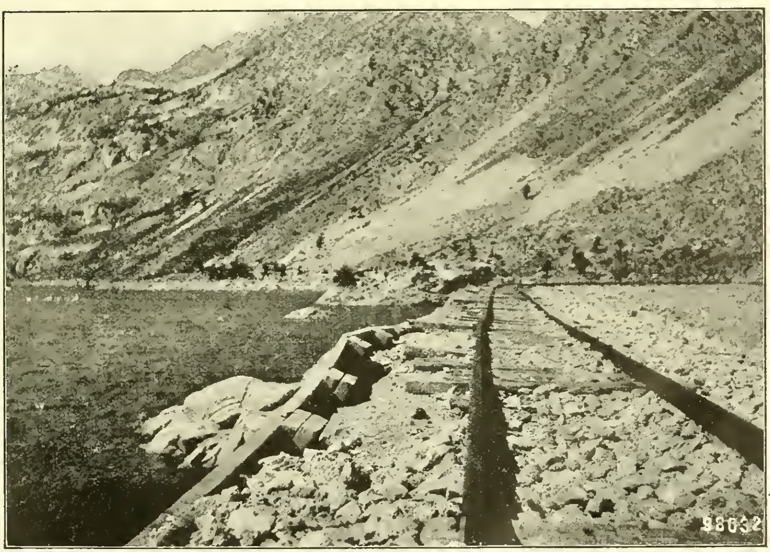


Fig. 14.—Crest of Timber-Faced Rock Fill Dam on Middle Fork of Bishop Creek, California, Showing Construction and Settlement.

water to which the rubble masonry was subjected before the concrete slabs were placed, and if any increase in leakage resulted. Mr. Galloway.

The problem of the water-tight skin is the most serious one to solve in dams of this type. A timber skin, consisting of three layers of planking, has been used on a number of dams. This is an expedient only adopted to save money. It is possible to make a water-tight skin in this way, and this was used by the writer in the design of two rock fill dams noted below. The placing of a diaphragm in the center, as in the Lower Otay Dam, is not believed to be good practice. The water-tight skin should be placed on the water face of the dam.

The use of reinforced concrete, as far as the writer knows, was first tried on an extensive scale at the Relief Dam, of the Sierra and San Francisco Power Company, in California. This dam was built in 1908 and 1909 under the supervision of Messrs. Sanderson and Porter, Constructing Engineers, of New York, with Messrs. C. D. Marx, Wynn Meredith, Donald Frye, and the writer as Advisory Engineers. The dam is 140 ft. high; Fig. 11 is a cross-section. In the opinion of Professor Marx and the writer, it was necessary to have a diaphragm of sheet steel embedded in concrete for the face. The other engineers believed that the concrete would withstand the pressure of water, and experience has proved the correctness of this idea, as the dam was built with a reinforced concrete skin 3 ft. thick at the bottom and tapering to 9 in. at the top.

Another point to which attention should be called is the necessity of having a substantial backing of derrick- and hand-laid rock behind the water-tight skin. In the Morena Dam this backing is 50 ft. thick at the bottom. In the Relief Dam the backing is 100 ft. thick at the bottom and 13 ft. thick at the top. The face behind the concrete is laid in cement mortar for a thickness of 2 ft.

The writer takes this occasion to call attention to what is likely to happen to an engineer when he does not superintend the building of a structure for which he has made the design. In December, 1905, the writer prepared plans of two rock fill dams for the Nevada-California Power Company, on Bishop Creek, California. The dams are at an elevation of about 8 500 ft., and are in a very remote part of the State. For reasons of economy, timber faces were designed. Each dam was about 75 ft. high, and the design followed closely that of the Bear River and Meadow Lake Dams of the Standard Electric Power Company, now a part of the Pacific Gas and Electric Company. These dams on the Mokelume River, also about 75 ft. high, were built by W. R. Eckart, M. Am. Soc. C. E., about 1899 and 1900. Fig. 12 is a cross-section of the dam on the Middle Fork of Bishop Creek as designed. The water face has a timber skin of three layers of plank. The design called for a backing of derrick-laid rock, as shown, on both the front and back of the dam.

Mr. Galloway.

In the strenuous times following the earthquake of 1906 in California, the writer severed his connection with the Power Company, and the construction of the dam on the Middle Fork, which had just started, was carried out by others. The derrick-laid rock was omitted on the Middle Fork Dam, with the result that settlement took place, distorting the timber face and causing considerable leakage. The settlement was as much as 3 ft. in some cases. Figs. 13 and 14 show the distorted sheathing.

In the description of the Morena Dam, Mr. O'Shaughnessy notes the necessity of having the quarry waste segregated from the loose rock fill at the back of the dam. A difference of opinion might exist on this point. The writer believes that if the fill be made of all sizes of rock, from quarry waste up, there will result a more compact mass, which is less likely to be distorted than if only large stones are used. The analogy is suggested by the compactness of a well-laid macadam pavement which resists distortion under exterior forces much better than rock of nearly uniform size. The statement that less damage would result from water leaking through the mass does not seem to justify the rejection of the quarry waste, as leakage, even if a considerable quantity of water passes, would not be harmful. There might be a local settlement, but, with the slopes adopted in the Morena Dam, it would not seem to be important. This remark refers to dams with a proper slope of the fill, and not to one of bad design such as the Walnut Grove Dam in Arizona.

The foregoing observations are not made as criticisms of the Morena Dam, for the writer believes it to be a structure of excellent design, on which Mr. O'Shaughnessy is to be congratulated for its planning and execution.

Mr. Hawgood.

H. Hawgood, M. Am. Soc. C. E. (by letter).—This paper is replete with matter which is interesting and useful to those engaged in water projects. Through the courtesy of Mr. O'Shaughnessy and Mr. Moloney, the writer was afforded an opportunity to examine the structure minutely, and can testify to the thoroughness which characterized the work and the methods used.

The dam embodies in a marked degree two features essential to successful rock fill or earth dams, that is, an impervious, or practically impervious, face, with a pervious back. The writer agrees with the author in regard to the advisability of keeping the rock fill, behind the water-tight skin, clear of soil and open to free drainage. In his opinion it is questionable whether such settlements as may occur will cause the masonry face to leak sufficiently to warrant the contemplated concrete slab facing. It is improbable that such leaks will be other than small, or have any effect on the integrity of the structure, and such being the case, the stopping of leakage at an expenditure greater than the commercial value of the escaping water, would not be justified, and particularly as, in this instance, all leakage, save such portion thereof as would be lost in evaporation in traversing the intervening cañon, would be recovered by the Barrett Dam, 10 miles down stream. An absolutely water-tight dam and reservoir is an academic ideal, rarely, if ever, attained in practice. Seepage, or leakage, is of no particular moment, provided its possibility has been fully safeguarded in the design, and its magnitude be insufficient to become of commercial importance. The "dry" gate-tower and tunnel outlet, independent of the dam itself, remove a fruitful source of trouble. Mr. Hawgood.

The writer spent some time in the cable engine-room watching the working of the signaling apparatus, and being engaged at the present time in building a dam where the signals are conveyed by bells, was impressed with the superiority of visual over audible signals. With bells it is a matter of count, memory, and wakefulness, as to how many bell taps were given. With the visual signals, the call remains illumined until replaced by another, thus eliminating any element of uncertainty, and minimizing risks.

It would be of interest, in connection with the water-shed and rainfall, to know the discharge capacity of the spillway, and it is hoped that Mr. O'Shaughnessy will supply this information.

The difficulty of deducing run-off satisfactorily from rainfall is well illustrated by the rainfall and run-off tables in the paper. With a precipitation of 12.79 in. at Barrett and 18.56 in. at Morena in 1907, the run-off was 2.62 times that of the succeeding year, with a precipitation of 16.82 in. at Barrett and 20.56 in. at Morena. Similarly, in Table 3, for the Sweetwater water-shed, 15.52 in. in the season of 1905-06 produced 2.54 times the run-off of the preceding season, with a precipitation of 15.55 in.

Applying Kutter's formula to the particulars of the Dulzura Conduit, as far as they are given, it would appear that the value of n approaches 0.02. It hardly seems probable that, with so smooth a conduit, the coefficient would be so low; perhaps Mr. O'Shaughnessy will throw some further light on this point.

F. B. Maltby, M. Am. Soc. C. E. (by letter).—This paper is especially welcome at this time, as the construction of many dams for various purposes and distributed widely over the country is in contemplation, and the disastrous results of some recent failures of such structures have brought into prominence questions relating to their design and construction. Mr. Maltby.

The writer regrets that Mr. O'Shaughnessy did not discuss more fully the reasons for choosing this particular type, and especially for the dimensions and slopes used. It is hardly necessary to open a discussion on the relative results of the different kinds of construc-

Mr. Maltby. tion: hydraulic fill, solid masonry, or rock fill. Each has its advocates, and its particular advantages in certain localities and under certain conditions.

It seems that, under existing conditions, cost of materials, etc., the type selected is probably the most suitable for the locality. The writer also thoroughly agrees with Mr. O'Shaughnessy as to the inadvisability of making a combination of a hydraulic and rock fill. Such a dam would have the particular advantages of neither type, and would possess the weak features of both.

One of the strongest arguments in favor of a rock fill dam, outside that of cost, is based on the feature of construction which permits of thorough and comparatively unobstructed drainage, and relieves the structure of the injurious effects of uplift due to leakage either through or under it, which was brought out so prominently by some recent discussion.

The writer does not see the necessity or desirability of the reinforced concrete slab facing. If he understands the drawings and description, the upper face of the wall, to a depth of 6 ft., was laid in mortar, and this wall certainly could have been made tight enough for all practical purposes. If, through settlement, cracks appear which permit an undue quantity of leakage, they can be filled or caulked at much less expense than by placing the proposed concrete slab.

It sometimes seems that, in the present age, when the use of reinforced concrete is advancing so rapidly, engineers are prone to introduce such construction unnecessarily. Even if it was desirable to cover the face of the dam with a concrete slab, the writer does not understand the line of reasoning used in determining the quantity of reinforcing steel.

As the slab is to be poured directly on the face of the wall, there is no unsupported space or span to be carried, and the only function of the reinforcing steel is to provide for expansion, and, as expansion joints are provided every 48 ft., the matter would not be a serious one.

If the steel is to provide for expansion only, why should there be more of it at the lower part of the slab than near the top? The change or variation in temperature at the top of the dam will certainly be greater than at the bottom.

Again, it is difficult to understand the utility or necessity of the anchor bars set into the backing on 4-ft. centers. Apparently, they extend into the backing about 3 ft. and into the slab about 3 in.

It is hoped that the author will state the reasoning used in determining the quantity of steel, especially as it must have been a very serious item of expense.

It is regretted that the author has not given as much and as exact information concerning the construction and detailed cost of the dam proper as is contained in the interesting description and cost of blasting the rock. Mr. Maltby.

M. M. O'Shaughnessy, M. Am. Soc. C. E. (by letter).—In reply to Mr. Galloway, it may be stated that the paper shows the concrete facing to have been discontinued at the 42-ft. contour, so that the water pressure to which the masonry without concrete facing was subjected was at the 65.33-ft. line, or 23.33 ft., and that the leakage diminished as the pressure of the water increased, due to the compression of the mud and silt in front of the old concrete toe wall, which was constructed of coarse rubble concrete in 1897-1898. The writer notes that in all the recent German masonry dams a layer of clay and soil is deposited on the water face of the masonry. Mr. O'Shaughnessy.

The writer concurs with Mr. Galloway in his objections to the central diaphragm in a rock fill dam, unless such dam is of the cellular type of reinforced concrete, and capable of inspection; then, if leakage occurs, it can be stopped by plugging up some of the leaky cells.

Mr. Galloway alludes to the Relief Dam and his connection with the original designs of that structure. Owing to the location of the bed-rock, it is believed that the original plans recommended by him were not followed out in construction, as it now presents a concave face up stream, which puts the concrete skin in tension instead of compression, and has induced many cracks which have developed serious leakage. This leakage, in time, is bound to wash through the dam the "fine material washed in with water to fill voids," which will cause subsequent settlement and impair the effectiveness of the water-tight face. The writer disapproves of the adoption of sheet steel in the concrete facing, as the adhesion of the concrete to the smooth face of the steel would be questionable, and, with an empty reservoir and no expansion joints, the temperature changes might cause the steel to buckle, thus creating a cavity which might be subjected to hydrostatic pressure, which would be contrary to the "Hydraulic Principle" expounded so clearly by Mr. Dillman.

Soil, silt, and clay were excluded from the mass of the dam, but not quarry "waste," composed of spalls, as alluded to by Mr. Galloway. In fact, there was a shortage of these materials, and, on the hand- and derrick-placed portion of the dam, numerous men with hammers were employed to break off the sharp edges of flat stones, and chink in the cavities with broken rock.

Mr. Galloway's and the writer's views, therefore, agree as to the benefit of using clean quarry waste, free from muck and soil, in the interstices of the dry rubble wall, and his remarks were no doubt caused by lack of clearness in the writer's description.

The spillway will have a capacity of 8 400 sec-ft., and will consist of a channel, 60 ft. wide and 5 ft. deep, on a 3% grade, with an inclined entrance 120 ft. wide, all cut out of the granite mountain side.

Mr. O'Shaughnessy.

The excavation for the dam was planned so that the materials from the spillway excavation were not wasted, but were put into the structure. The entrance to the spillway will be controlled by twelve radial gates, 8 ft. 6 in. wide and 6 ft. high, operated by a crab which runs on a track above the gates, as shown on Plate III.

The heaviest floods, measured on the Cottonwood lower down at Barrett, from 250 sq. miles of water-shed, have been about 7 000 sec-ft., and, as the Morena Reservoir has a capacity of 15 000 000 000 gal., for a water-shed of 135 sq. miles, the writer feels secure in the safety of the spillway provisions.

In reply to Mr. Hawgood's further query with regard to the Dulzura Conduit, to which the writer generally referred as being 5 ft. wide and 4 ft. 2 in. deep, with side slopes, etc., a typical section has a top width of 6 ft. 5 in., a bottom width of 3 ft. 1½ in., and a depth of 4 ft. 5½ in. On March 10th, 1911, with a depth of water of 3.3 ft., the wetted perimeter was 15 ft. 8⅝ in., the water area was 14.335 sq. ft., the hydraulic radius, 0.918 ft., the slope, $S, = 0.0008$, and the measured discharge over a wier was 40 187 866 gal., which would make n, in Kutter's formula, 0.016, which agrees very closely with previous recognized values of this factor.

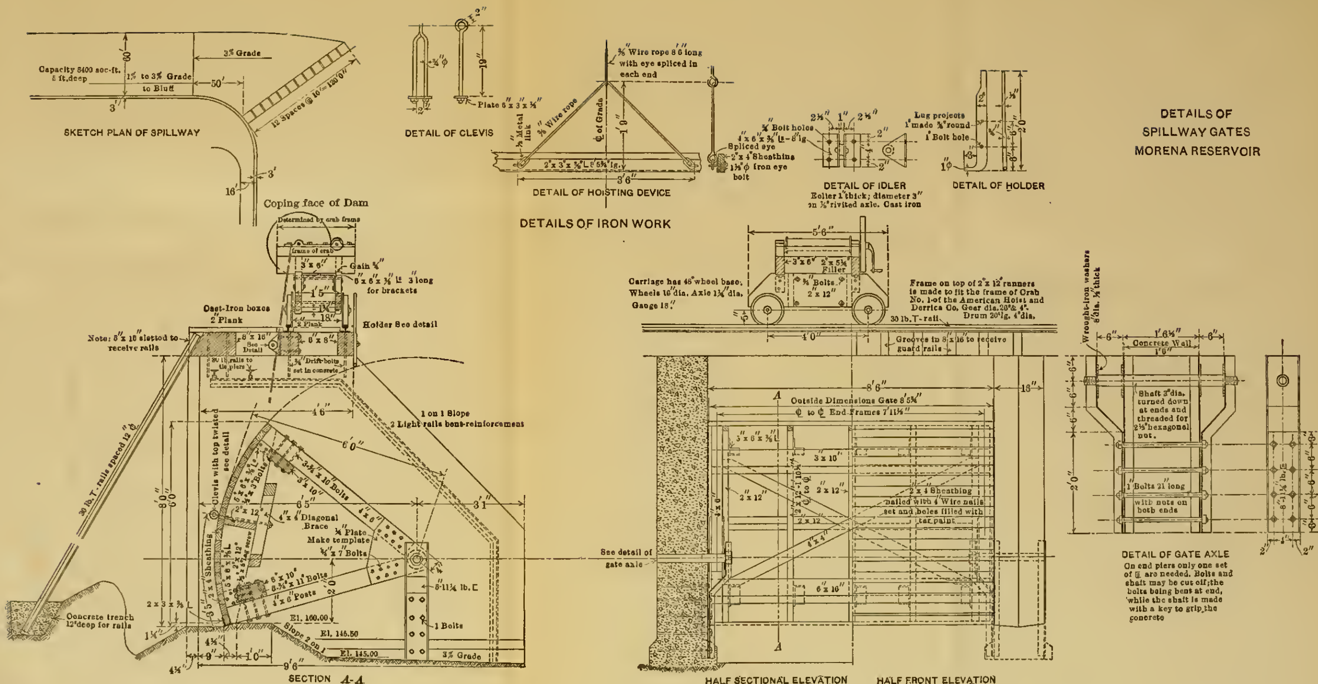
As the conduit at present draws water from the Pine and Cottonwood stream beds, and they contain very much fine mica and silt which escapes past the sand and scouring chambers near the entrance and floats along the bottom, the writer never expects it to carry much more than 40 000 000 gal. per day.

In response to Mr. Maltby's request for further reasons for slope selections on the dam construction: The original water slope was intended to be 1⅓ horizontal to 1 vertical, by the parties who in 1896 projected this structure, but it only reached as far as the toe wall on the 30-ft. contour in 1898. It was at that time intended to put on a water-tight face of asphalt concrete. Such a face could only survive on a flat slope, and as asphalt with the ultimate evaporation of the volatile oils was not considered by the writer the best material to use for this purpose—for undoubtedly periods will occur when the run-off will not be large enough to fill the reservoir, and the surface of the dam will be exposed to intense summer heat and its consequent destructive influences—this method was abandoned and the present type of water-face adopted. It is apparent that, the nearer the water-face approaches the vertical, the smaller and more economical will be the contents of the dam, so that a slope of ½ horizontal to 1 vertical would be ideal for building a masonry rubble wall.

Owing to doubts about the methods adopted in building the old fill behind the toe wall, and the character of this fill, as earth and soil, at the time of that construction, were dumped indiscriminately with rock, parts of which the writer had to remove, he decided to adopt the

PLATE III.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXV, No. 1218.
O'SHAUGHNESSY ON
MORENA ROCK FILL DAM.

DETAILS OF
SPILLWAY GATES
MORENA RESERVOIR



Mr. O'Shaughnessy.

9 horizontal to 10 vertical slope up to the 120-ft. contour, and thence to the top the ½ to 1, or preferable, slope.

The base of the old work was started much wider, owing to the intended flatter water slopes, hence the 21-ft. berm on the back at the 100-ft. contour. If the dam is ever raised, through the desire for more capacity, because of the reservoir silting up, or for other reasons, this berm can be well utilized for this purpose, thereby reducing the expense of raising the structure.

The lower slope of all rock fill dams should be 1½ to 1, or about the natural slope of the rock, and a rubble wall, or the hand-placing of the rock in this portion of the dam, as in some structures alluded to in the discussion, is thought by the writer to be an unnecessary expense and refinement, except for esthetic purposes.

As stated in the paper, the reinforced concrete was discontinued at the 42-ft. contour, and the greatest care was taken to make the masonry face tight and of first-class construction from this level to the top, all large face stone being washed with a hose jet before being bedded in the cement mortar, and the latter being tamped into all the joints with iron spoons. The views of Mr. Maltby and the writer are in agreement, therefore, as to the effectiveness of this method of construction, though it is more expensive than the concrete face work.

The object of reinforcing the concrete facing was to prevent cracks, so that each 48-ft. section would be a unit slab, firmly attached to the masonry, but free to move at the joints in response to any temperature or settlement stress. The anchor rods through the masonry form an effective bond between it and the reinforcing rods, and are also useful in construction operations in fixing accurately the position of the bars.

The apparent excess of steel in the foundation face work is a precautionary measure for the purpose of making an effective bond between the new concrete and the old foundation toe wall of rubble concrete built 12 years ago.

This toe wall was 12 ft. wide at the 30-ft. contour, and 4 ft. from the water face had a slot, 1 ft. wide and 5 ft. deep, into which the grillage of steel bars is nested.

As the work was not completed at the date of writing the paper, the writer refrained from giving the detailed costs desired by Mr. Maltby, and as there are many elements, such as construction roads, telephone lines, insurance costs, interest during construction, etc., which must be computed before the entire cost is obtained, such information might be misleading. Approximate estimates are available, however, which show the cost of a dam of this type in a very favorable light, compared with a masonry dam such as the Roosevelt, in Arizona, which it closely resembles in size, as Table 8 will show.

As the writer had much difficulty in procuring any reliable pub-

Mr. O'Shaughnessy.

lished data describing mass shots, he took advantage of this opportunity to publish the Morena results, in the hope that they might be of interest and value to his brother members.

TABLE 8.—COMPARISON OF ROOSEVELT AND MORENA DAMS.

	Roosevelt Dam.	Morena Dam.
Height	280 ft.	267 ft.
Thickness of base	170 "	300 "
Thickness of top	16 "	16 "
Crest length	1 080 "	550 "
Contents	340 000 cu. yd.	306 000 cu. yd.
Cost	$3 468 000	$1 100 000
Time consumed in building	4 years.	5 years.

In connection therewith, it is interesting to note the erroneous ideas which prevailed regarding the costs of structural work in San Diego as late as 16 years ago. The writer quotes verbatim the preliminary estimate, made at that time by a San Diego hydraulic engineer, who has since obtained distinction in his profession, even to the extent of writing a book on dams and reservoirs:

"Estimate of the cost of a rock fill dam 125 ft. high, or 5 ft. above the line of 1 000 in. capacity, is about as follows:

"Estimate of Morena Dam, 120 ft., Rock Fill.

"Preparation of foundations, concrete in base and toe walls, etc.	$5 026
Three outlet pipes, laid in trench cut in solid rock and bedded in concrete, with valves, etc.	9 500
160 000 cu. yd. rock fill, 80%, put in place by powder at 10 cents per yard	12 000
20%, or 40 000 yd., picked up and placed by ropeway at 40 cents	16 000
6 680 cu. yd. dry rubble wall at $2.50	16 700
30 666 cu. ft. asphalt concrete face at 50 cents	15 333
Spillway and gates, capacity 25 000 cu. ft. per sec.	20 000
Total	$94 559"

While the writer believes that reinforced concrete possesses merits in many locations and for many purposes, he is also a believer in its limitations. The permanency of any structure made of it will depend much on high-class manipulation of the ingredients, as well as thorough proof of the reliability of the cement, and the character of the sand and rock used. The American cement industry has expanded to such an extent in recent years that engineers should observe the greatest caution in using any new or untried brands before time has demonstrated their worth.

The writer believes a reading of the paper and discussion will have developed the following conclusions: Mr. O'Shaughnessy.

1. That a masonry dam, with a wagon-hauling cost of 1 cent per lb., or $4.00 per bbl. for cement, would have been more expensive than the present structure.
2. That freedom from uplift pressure is a desirable feature in favor of the rock fill type.
3. That the great "Hydraulic Principle" of one impervious surface next to the water pressure, as elucidated by Mr. Dillman, is the object to be obtained by engineers in dam construction.
4. That rock can be excavated economically by mass shots at higher levels above a dam without endangering the site formation, provided the strata are located so that the effects of the explosion will not open seams in the vicinity.
5. That the circular outlet tower of the type designed is the most economical that can be constructed.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1219

NOTES ON A TUNNEL SURVEY.*

By Frederick C. Noble, M. Am. Soc. C. E.

With Discussion by Messrs. George D. Snyder, Robert Ridgway, B. F. Cresson, Jr., S. M. Purdy, Lazarus White, and Frederick C. Noble.

The following is a description of the surveying methods used in constructing the tunnel under the East River between South Ferry, Manhattan, and Joralemon Street, Brooklyn. This tunnel was begun in March, 1903, and was opened to traffic in January, 1908.

The tunnel consists of two parallel single-track tubes, of the usual cast-iron ring construction, having an interior diameter of 15 ft. 6 in. Under the river their distance from center to center is 28 ft., and under Joralemon Street it is 26 ft.

The Manhattan headings were driven from a double shaft in Battery Park, near the ferry. Rock was encountered until the headings reached the lowest point of grade, near the middle of the river, after which the advance was continued with shields. The Brooklyn headings, for which shields were used from the start, were driven from two shafts, one over each tube, situated in Joralemon Street about 1 200 ft. back from the water front, at the point where the tunnel grade rises above water level. Under Joralemon Street the material encountered was rather coarse sand, containing gravel, cobbles, and boulders. Under the

* Presented at the meeting of February 7th, 1912.

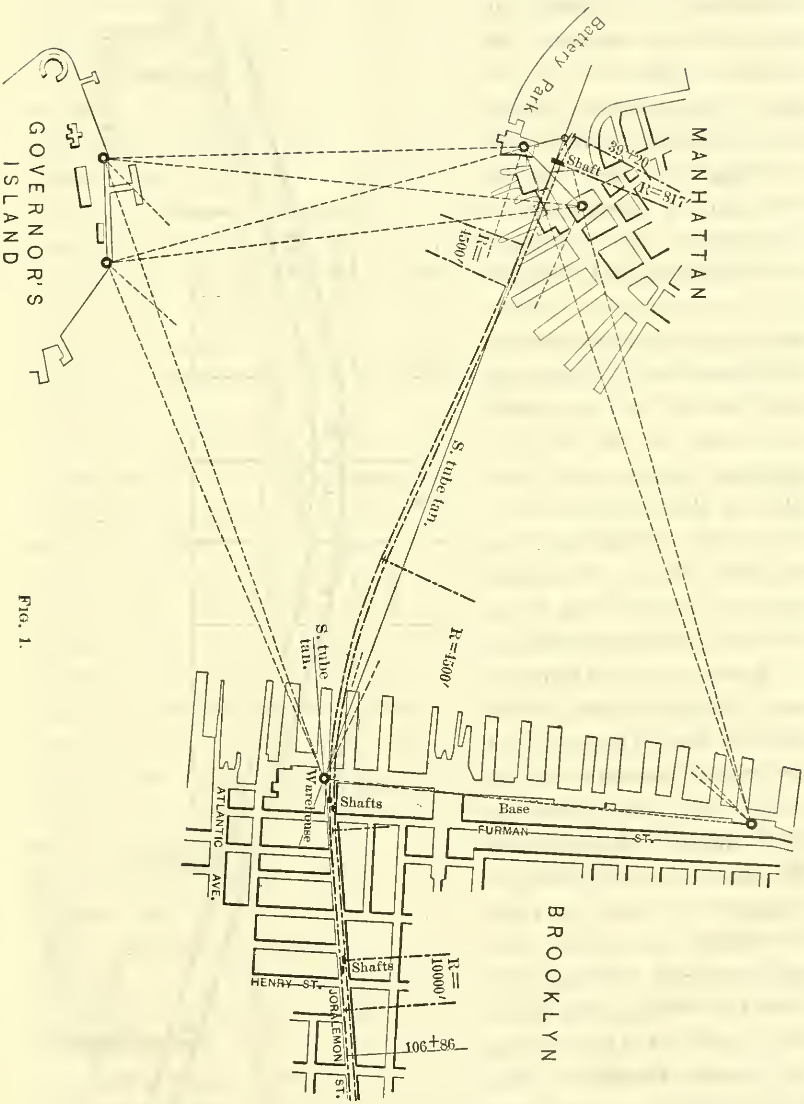

GOVERNOR'S ISLAND
MANHATTAN
BROOKLYN
Battery Park
Shaft
R=817'
R=4500'
S. tube tan.
R=4500'
S. tube tan.
Warehouse
Shafts
Base
FURMAN ST.
ATLANTIC AVE.
Shafts
HENRY ST.
JORALEMON ST.
R=10000'
106+86

Fig. 1.

river, except for a few hundred feet where the Brooklyn shields were slid through upcropping rock, the sand was extremely fine and of the same nature as the quicksand generally found in excavating for deep foundations in the lower part of Manhattan. The meeting of the Manhattan and Brooklyn shields took place in the fine sand midway between the two rock formations. The tunneling was all in compressed air, except in the Brooklyn headings above water level east of the shafts, and except inside the bulkhead line at South Ferry. The maximum air pressure was 42 lb. per sq. in. above atmosphere.

Between the Manhattan and Brooklyn shafts, a distance of about 6 000 ft., there are three tangents in the tunnel line, the middle or river tangent being joined to the Manhattan and Brooklyn tangents by curves of 4 500 ft. radius. (Fig. 1.) The grade on each side is 3.1%, joined at the middle by a short grade of 0.5% sloping up toward Brooklyn. The grade tangents are connected by vertical curves of 20 000 ft. radius. (Fig. 2.)

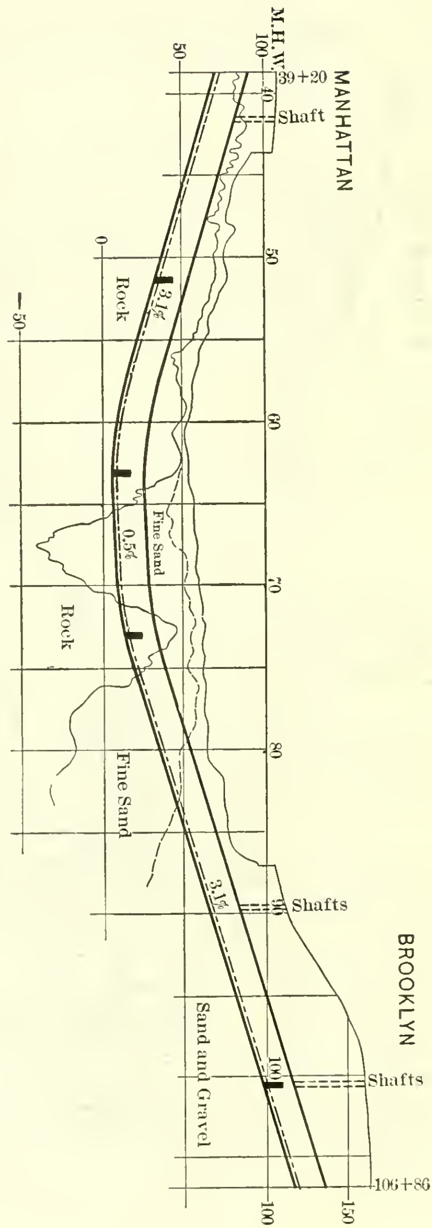


Fig. 2.

The intersection of the south tube lines of the Joralemon Street tangent and the Manhattan tangent produced was located on the roof of a dock warehouse at the foot of Joralemon Street. It was necessary to determine the exact distance between this intersection and a point of known stationing on the south tube line in Manhattan, and this required a triangulation survey. It was apparent that no base line of suitable length could be laid out in the vicinity that could be measured directly from end to end as one of the sides of the triangulation system. Although the ends could be located on roofs, so as to be seen from each other, the measurement had to be made along a broken line. A reconnaissance showed that a practicable base line traverse, about 3 000 ft. long, might be obtained on the Brooklyn water front north of Joralemon Street. The southern terminus was chosen to be the abovementioned intersection of the south tube lines, and the northern terminus was also located on the roof of a warehouse. Each of these points commanded a view of each other and of the opposite water fronts of Manhattan and Governor's Island, and, accordingly, were suitable for triangulation stations. In Manhattan, because of intervening buildings, it was found impracticable to locate a triangulation station on the south tube line in the vicinity of the shaft so that it would be visible from all the other stations. On this account two elevated points on the roofs of near-by buildings, one on each side of the Manhattan tangent, were chosen as triangulation stations, and carefully referenced to the point of known stationing on the south tube line, which was visible from them. On Governor's Island, on the northerly sea-wall, two triangulation stations were chosen which marked the extremities of a shorter base used two seasons before in making a preliminary triangulation of the tunnel line. It was desired to include this base in the system so as to have an independent check on the Brooklyn base measurement.

The foregoing arrangement gave four fairly well proportioned quadrilaterals, the Brooklyn base forming one common side. In the system there were no angles of less than 26° or more than 78°, and the ratio of length of base to the distance sought was about $1:1\frac{1}{2}$. By having four quadrilaterals from which to calculate the co-ordinates of the Manhattan points, referred to the Brooklyn base as the axis, it was thought that any material error in the angular work could be detected, and this would give a measure of the error to be regarded as unavoidable.

The base line traverse was measured with a 50-ft., flat, steel-wire tape, provided with a spring-balance handle containing a level bubble, thermometer, and adjustment for temperature correction. It was marked with nicks at each end for use with plumb-bobs. The tape was tested for absolute length by comparison with a standard tape of known error, as determined by the Bureau of Standards at Washington. Measurements were made on cloudy days, or during early mornings in the shade, to secure uniform temperature conditions. The line was marked in advance every 50 ft., and paper pads, ruled with a longitudinal line, were weighted down at these points, so as to relieve the head chainman of the distraction of getting line from a transit. Each measurement was made by simultaneous plumbings at the ends of the tape, which was held level. The forward point was marked at the average of a number of trials. For each series of measurements a fresh sheet of the pad was used. With a tape of this kind the difficulty of steady and precise plumbing was such that no individual measurement could be depended on to less than 0.003 ft., or 1 in 17 000. No doubt a better method consists in using a longer tape, supported at short intervals, with a tape-stretcher, and measuring directly, without plumbing, on portable stations, as was done later in connection with other tunnel surveys in New York. However, with care, it was found that the errors were not cumulative, and, in a distance of 2 880 ft. measured in this way, the variation between the mean and the greatest extreme of several measurements was only about 1 in 100 000. The ends of the line measured on the docks were connected to the triangulation stations on the roofs in two different ways, in order to avoid repeating an error. The points on the copings were transferred to the dock level by casting down intersecting lines of sight from three transit set-ups. As a further check, an independent traverse, closing on the same points, was run by a different party in the marginal street back of the warehouses, and this gave a result agreeing with the dock traverse to 1 in 150 000.

The triangulation sights were octagonal pine poles, $2\frac{1}{4}$ in. in diameter and 8 ft. high, painted in alternate bands of red and white. While these could be bisected readily, and gave no sensible difficulty from phase, because of the comparatively short range and the cloudy weather selected for instrumental work, it would have been better practice to use higher targets with wide, flat vanes, set alternately at right angles.

The angles were read separately by two observers, with different instruments. These were ordinary engineers' transits, reading to 20″ on a $6\frac{1}{4}$-in. limb. The eight angles of a quadrilateral were measured separately by successive additions upon the limb. The entire angle at a station was also read, as a check on the sum of its components. To avoid bias, the angles were started at random near zero. Turnings were made, accumulating from left to right, until the sum became as nearly as practicable 360°, or its multiple. Reversing the telescope, and setting on the right-hand target, the same number of turnings was made from right to left until the sum was diminished to near the original reading. Both verniers were read at the beginning, at the middle, at intermediate points, and at closing. In case an angle failed to close nearer than an average of 4″ to each turning, the result was discarded and the angle was read again. When the third angle of any triangle of the system had been read, the angles were added, and if their sum varied from 180° by more than an average of 5″ for each angle, the three angles were read again. The quadrilaterals were tested in like manner. It was found necessary to read over only one triangle, and in this case the trouble was located at a station where the conditions for observing had been less favorable than elsewhere.

In comparing the observed angles, it was noted that the direct reading of an angle was generally greater, by 2″ or 3″, than its value as found by taking the difference between two angles, and that, at the end of an observation, the closing reading was nearly always greater than the first. As this pointed to a small persistent error, it was seen that it would have been a better programme to measure each angle both directly and by its 360° complement, and to take the mean.

The set of observed angles of each quadrilateral was adjusted by the method of logarithmic residuals, based on the principle of least squares, according to Johnson, and using the rigid method of making the side equation correction. The angles read by each observer were adjusted separately. The corrections for spherical excess and reduction to sea-level were omitted as insignificant.

The four quadrilaterals formed four pairs, each with a common triangle. These relations would give rise to additional equations of condition, and the simultaneous adjustment of all four quadrilaterals would be possible, theoretically. However, this was not attempted because the refinement would not pay for the effort, and because it was desired to compute each quadrilateral separately adjusted, so as to gain a practical idea of the effects of error in the angular work.

For the purpose of comparing results, the co-ordinates of the Manhattan and Governor's Island stations, referred to the Brooklyn base, were then computed for each of the four adjusted quadrilaterals of each observer. On plotting to full size it was found that for any point the various positions thus computed fell within the section of the target pole. It was seen that considerable time and labor would have been saved (and with sufficiently accurate results) by using any one quadrilateral, but the advantage of learning in this way the probable extent of errors from all causes was thought to warrant the extra trouble.

The sides of the triangle formed by the two Manhattan triangulation stations and the point of known stationing in Battery Park were then measured, and the co-ordinates of the latter point were computed from those of the other two. The distance triangulated for was then determined. The computed length of the Governor's Island base, about 700 ft., was found to agree with its original measured length within 0.016 ft., or much less than the probable angular error would account for.

The stationing of the Brooklyn intersection of the south tube tangents then being known, base-tape measurements were made from this point to the Brooklyn shafts. Stationing was transferred to the tunnel by plumbing in the shafts. All measurements, above and below ground, to determine stationing in the tubes, were made with the same kind of tape as that used for measuring the base, and in a similar manner. When the first headings were connected, the measured distance was found to be about 0.2 ft. short of that calculated. The discrepancy was somewhat more than had been expected, but may be adequately accounted for as combining the error in angular work with the error due to the difference in conditions, with the apparatus used, between taping in the tunnel and taping the base line on the surface. Its effect on the results of the surveys for alignment and grade, however, was very slight.

For establishing the Manhattan tangent alignment, there was no point on line in Battery Park, near the shaft, that was high enough to afford a sight to Brooklyn over the intervening ferry-houses. For this purpose, two towers, about 35 ft. high, one on each tube line, were erected about 150 ft. west of the shaft. From these it was possible to look across the river over all obstructions and see targets on the warehouse roof in Brooklyn set on the Manhattan tube lines prolonged. The towers were built as triangulation towers usually are: with an interior tripod of three 8 by 8-in. timbers rigidly bolted together at the top and braced; this being enclosed within, but nowhere touching, a four-post tower braced on all sides and supporting a platform for the observer a little below the top of the tripod.

Line was transferred to the tunnel in the following manner: A transit, mounted on a trivet, was set on the tower tripod on the tube center line, as plumbed up from a hub under the tripod by several reversals of another transit set on the center line a short distance away. The tower transit, fore-sighting on the Brooklyn target, set, by repeated plungings, a point on center line on the surface, about 10 ft. beyond the shaft. On this point a transit was set up, with its plumb-string in the mean position of the several settings, and, fore-sighting on the plumb-string of the tower transit, aligned two plumb-wires suspended in the shaft.

Steel piano wire, $\frac{1}{50}$ in. in diameter, was used for this purpose. It was wound on brass reels 9 in. in diameter. The reels were set, with their axes across the line, on stringers attached to the head-frame posts, high enough to permit sighting on the wires and to fore-sight by merely changing the focus. As the wire left the reel it ran over a small grooved wheel on a threaded axis, turned with a thumb-screw, by which slow lateral adjustment could be given. The wire was kept taut by a 20-lb. weight suspended in a pail of water at the bottom of the shaft.

The wires were shifted laterally until the vertical hair exactly bisected the plumb-string fore-sight and both plumb-wires. When this condition was reached, a transit, set up in the tunnel below, as close to the wires as practicable, was shifted laterally until its vertical hair exactly bisected both wires, and reversing, threw the line forward, setting two or more plummet-lamps hung from verniers reading on graduated brass scales attached to the roof of the tunnel. Resetting on the wires, another set of readings was taken. Two more sets were taken in like manner with telescope reversed. The wires were then shifted and reset by the upper transit, and the previous operations were repeated for four more sets. All the foregoing operations were then repeated, after interchanging transits at top and bottom, turning the water pails containing the weights partly around, shifting the wires, examining them to see that they hung free, and interchanging the members of the party. This work of plumbing the lines down the shaft was usually done on Sundays, when the cages were not running.

At the Brooklyn shafts, the same method was followed, but the first part of the operation was simpler because the warehouse targets could be sighted directly from the surface at the shafts.

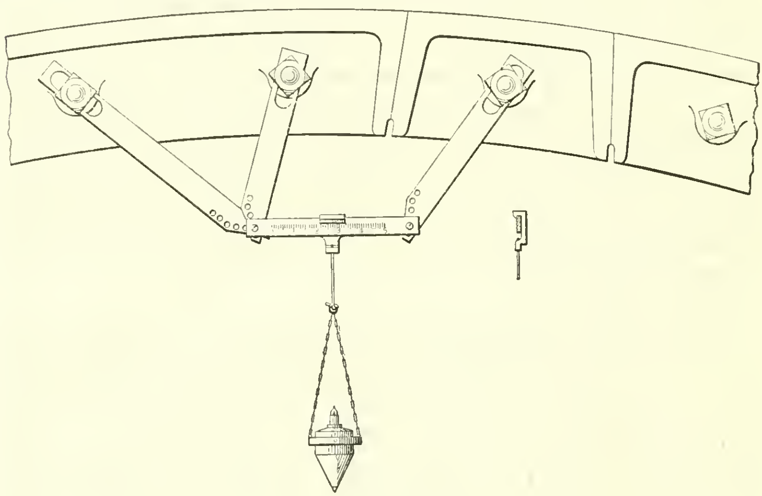


Fig. 3.

The base between the wires was about 10 ft. at the Manhattan shaft and 15 ft. at the Brooklyn shafts, and in each case was as long as permitted by the width of the shaft. On account of the shorter base in Manhattan, and the extra operation there in transferring line, opportunity was taken, when the headings were afterward extended west from the shaft, to check the line by plumbing through a vertical 6-in. pipe sunk from the surface to the tunnel at a point on the center line of one of the tubes, about 130 ft. from the shaft.

The alignment scales were brass bars, 1 in. wide, $\frac{1}{4}$ in. thick, and 10 in. long, graduated to 0.01 ft. They were connected by three flat iron bars to the bolts of the tunnel lining at the roof. (Fig. 3.) The scales were read to 0.002 ft. by a sliding vernier from which a plummet-lamp was suspended.

The line established on the scales near the shaft, as the mean result of many days' droppings, was produced by repeated runs read on scales set at intervals of about 200 ft. These runs were made by different observers, and using different transits, until mean readings were well established on the advance scales. Where the tube was on a curve, the scales were placed on tangents as long as could be obtained, and usually near points of intersection. The angles at these points were carried forward by repeated turnings on the forward scales.

When it became necessary to carry the line through a lock, the transit was placed on a trivet on a timber wedged across the lock at the forward end, where it had the support of the bulkhead wall. It was then lined in with the last two scales, the forward one having been located as close as practicable to the lock, and a reading was taken on a paper scale set inside the lock over the outboard door. The lock was then taken in, and, before throwing the line ahead, the paper scale was read again to detect any possible movement due to change in pressure. No trouble was ever experienced on this account, however, and when the bulkheads were advanced the lines thus locked through checked quite satisfactorily with lines carried through in free air.

To assist in aligning a transit with two given points, a very useful device was furnished by the instrument maker. This consisted of two sliding plates, which were attached between the transit base and the tripod head or trivet, and permitted considerable range of lateral movement by a slow-motion screw.

Grade was established between the two sides of the river as follows: Level runs were made from the shafts on each side to benches under the Brooklyn Bridge on the river side of each tower, at which point the river is much narrower than at the tunnel line. Levels were set up on each side, and simultaneous reciprocal readings were taken a number of times. The instruments were then interchanged and the readings repeated. In this way errors due to refraction, curvature, and imperfect adjustment were compensated.

Levels were transferred down shafts by reading on a tape hung in the shaft, and applying the required corrections. To guard against errors of method in using a tape in this way, a test was made by hanging the tape in the stair-well of a high building and comparing elevations obtained thus with those obtained by pegging up and down the stone steps with level and rod. Tunnel benches were generally located on lugs or cross-flanges near the lower quarter point of the tube.

Conditions for instrumental work in the tunnel were at times as good as could be desired, but at other times fog, powder smoke, and heat interfered seriously with the work. Much trouble was caused by scales being removed or disturbed, and, where this was likely to occur, they were boxed around. The zero points of scales were referenced to chisel cuts in the flanges of the lining. Where the tubes were in soft material, with one considerably in advance, the passage of the second shield would disturb the scales in the other tube.

When the Brooklyn headings had progressed about 1 000 ft. along the tangent, and part way around the curve to the foot of Joralemon Street, caissons were sunk at this point and connected to the tops of the tubes to form new shafts. To locate the corners of each caisson and to guide it during sinking, an underground traverse, tying in the corners of the roof opening, provided for by special segments, was reproduced on the surface. On making the connection the instrumental work checked closely.

These shafts afforded an opportunity to check the survey work thus far run, directly from the surface, and this was done with very satisfactory results. As the headings advanced on either side of the river, cross-connections were made between the tubes at certain points, giving an opportunity to compare the alignment, levels, and measurements carried forward in one tube with those carried forward independently in the other. There were three of these connections: two in rock on the Manhattan side, and one in the rock reef on the Brooklyn side.

The north headings were the first to meet under the river, and this was in December, 1906. When the shields were still 90 ft. apart, a 6-in. pipe was driven from the Brooklyn to the Manhattan side, by jetting and jacking, and the lines and grades were checked through. The pipe was driven straight enough to afford a clear sight from end to end, and, although somewhat off center line, was almost perfectly level, so that readings for grade could be made through it. A transit set up at each end produced the line through the pipe to the two forward scales in each heading, by which means the two center lines were found to be exactly parallel and 0.13 ft. apart. Of this discrepancy, about one-fourth could be attributed to the discrepancy of 0.2 ft., before mentioned, between the triangulated and measured distance across the river. The grades checked through the pipe within 0.01 ft.

The south headings, which had been keeping some distance behind the corresponding north headings, met in March, 1907. As the south shields were approaching to a junction, it was possible to reference the center line of each heading to the joined and corrected center line of the north tube. This was effected by pushing rods of a measured length through grout holes in the south side of the north tube until they brought up against the shell of the south tube at points which were then located and measured to. In this way the center lines of the south headings were brought together with an error of only 0.01 ft.

The greater part of the instrumental work in the tunnel consisted of giving the contractor lines and grades for the guidance of the shields and rock headings, and for the erection of the cast-iron lining of the tubes. A profile of the center line, taken at the top on the flanges of the rings, was kept posted up on a cross-section sheet in the contractor's office near the shaft, and a similar record was plotted for line. Where the lining was erected behind a shield, the transit and level were used directly for giving working lines and grades, and no simpler method could be devised that would answer the purpose, because it was found that the reaction of the shield jacks would disturb the lining for a considerable distance back. Where the headings were in rock, working points were given by lining in with the eye the flames of two plummet-lamps, hung from the roof by chains of adjustable length, and set so as to indicate grade as well as line. These lamps were made of a short piece of iron pipe, capped at the bottom, reduced at the top for a wick, and provided with a bail. They were suspended from hooks screwed in roof plugs, timber caps, or wedges in caulking recesses, with a piece of sash chain and an adjustable section made of two strips of brass, by which the length of suspension was varied. This arrangement was set by the line and grade party during the day shifts, and was ready for use by the inspectors at all times for erecting the rings, for giving points in rock headings, and for setting roof timbering.

DISCUSSION

Mr. Snyder. George D. Snyder, M. Am. Soc. C. E.—This paper forms an interesting contribution to the literature of a class of underground surveying on which very little has been published.

The object of such a survey, of course, is to fix the location of the tunnel accurately, and then to give the workmen sufficient information to enable them to build it in the pre-determined position, and also to enable the relation between points or objects on the surface and points underground to be determined. In this case, the subaqueous tunnel forming part of a longer underground railway, it was also desired to extend the continuous stationing of the land tunnels across the river; but the desire to ascertain this distance was not the only object in making a triangulation survey. All that was requisite was to connect the triangulation system with the land surveys by any convenient method, and then, by the aid of co-ordinates, to compute the length across the river on the center line of either tunnel.

In the description of the triangulation system, the paper refers to co-ordinates, but it is not clear that they were used throughout the underground surveys. The use of co-ordinates facilitates the computation so much, in the determination of the relations between points on the surface and underground, and in the determination of the objective in driving, that attention should be called to the fact in such a paper.

When a tunnel is to be driven in a straight line, on a level or slight grade, between shafts which are in view of each other, no elaborate or precise triangulation system is necessary, as the line on the surface between shafts can be transferred underground and projected ahead without having the exact distance between shafts, and this can be determined accurately enough for all practical purposes by a very simple triangulation. However, where curves are used, and where the shafts cannot be located on the axis of the tunnels, or where obstructions prevent a line of sight between shafts, a more or less precise triangulation survey becomes necessary.

The tunnels of the Hudson and Manhattan Railroad having been constructed contemporaneously with the Battery tunnels, some description of the survey methods followed on that railroad may prove of interest. This system consists of about 17 miles of underground railway (measured as single track), of which 12.4 miles are in driven tunnel, including four single tunnels crossing the Hudson River.

When the present company started construction in 1902, the work already done by the old company on the up-town tunnels opposite the foot of Morton Street, Manhattan, consisted of shafts on each side of the river, 3 916 ft. of the north tunnel constructed from the New Jersey

PLATE IV.
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side and 160 ft. from the New York side, and 570 ft. of the south tunnel built from the New Jersey side. The south tunnel, the work on the New York side, and 2 000 ft. of the north tunnel, were lined with brick. The remainder of the north tunnel was lined with cast iron with a shield. Mr. Snyder.

The first work done was the completion of the river tunnels, which were driven entirely from the New Jersey side. A base line was measured on the New Jersey side, with a triangulation tower at each end, the relation of the two shafts was found, the position of the old work was located by underground surveys, and the azimuth of a line from the old shield to the objective on the New York side was obtained.

As the work progressed it was felt that there would be more confidence in the accuracy of the surveys if a more careful triangulation was made. Therefore the triangulation system indicated by the points, *Y, X,* and *W,* Plate IV, was selected, and a base was measured along West Street on the New York side.

In the meantime surveys were under way for the down-town system of tunnels, the triangulation system being indicated by the points, 1, 2, 3, and 4, Plate IV. When the construction of these tunnels was authorized, the two triangulation systems were connected by a precise traverse survey indicated by the Points, *A, B,* and *C,* and a common system of co-ordinates was established, the origin being 2 000 ft. south of Point 5 and 8 000 ft. west of a meridian passing through this point, this meridian being approximately north.

The fundamental base for the up-town system was between Points *X* and *W*. Both these points being on top of buildings, the direct measurement of the distance between them was not practicable, but the measured base being on West Street, and approximately parallel, the relation of the triangulation points was obtained by turning angles to the triangulation points from points on a short base at the ends of the main base line.

The fundamental base for the down-town system was between Points 1 and 3, which were placed so that direct measurement could be made. A check base line was measured on the Jersey City side between Points 2 and 4. The ends of these base lines, as well as intermediate points, were marked by concrete monuments in which were embedded brass plugs with hair-line crosses. The length of the base line between Points 1 and 3 was 3 633.4 ft., and the longest sight in the entire system was 8 246.8 ft. Azimuth increased clockwise from the south point throughout the 360° of arc.

The transits were similar to those used by Mr. Noble, with $6\frac{1}{4}$-in. horizontal circles, reading to 20″, although some $7\frac{1}{2}$- and 8-in. instruments, reading to 10″, were used. There was a difference in regard to the cross-wires, however, for, instead of having a vertical wire, Mr. Snyder. there were two wires forming an angle of about 70° and intersecting slightly above the horizontal wire. The advantage of this arrangement is that the diagonal wires do not cover the plumb-lines which are used for sights. Inverting telescopes with large 1½-in. object glasses were used. The instruments had three leveling screws.

The base lines were measured as follows: Steel tapes 100 ft. long were used, and were compared with a tape which had been standardized by the United States Bureau of Standards. The line was prepared by marking points at intervals of about 98 ft., and at least two lines of levels were run over these points. Heavy movable spiders, Fig. 4, having a hair-line cross cut on the brass head, were then placed at these points, four being used in succession. The tape was then stretched between the spiders. One end was clamped to a weighted standard and the other end was given a pull by a 12-lb. weight attached to a cord passing over a ball-bearing pulley. The tape was supported at intervals of about 20 ft., these intermediate supports being set to a uniform grade between adjacent spiders. Readings were taken simultaneously at each end, and the difference gave the distance between the spiders. Ten observations were made, and after each reading the position of the tape at the spiders was changed longitudinally. Measuring with a rule the distances of the tops of the spiders above the points on which elevations had been taken, enabled the inclination of the tape to be determined, from which the horizontal equivalent was obtained. Each base line distance was the corrected mean of at least three such measurements. The observed distances were also corrected for temperature and for the constant errors of the tape. All measurements were taken at night, so as to avoid traffic difficulties and have more uniform temperature conditions.

The main triangulation angles were repeated at least 40 times, readings being made on the tenth repetition, and the telescope reversed after each successive ten repetitions.

Angles in the primary triangulation were first adjusted by the rigid method, according to Johnson. It was afterward considered that the base lines had less likelihood of error than the angles and a second adjustment was made so that identical results could be obtained in computing the sides of the triangles from either base of the quadrilateral.

As these tunnels form part of an interstate railroad, it was necessary to determine the portions in each State. The State line is approximately in the center of the river, having been established definitely by a Joint State Commission and referred to prominent objects on the shore, such as Trinity Church spire and the Bergen Dutch Reformed Church, which were also points of reference of the United States Coast and Geodetic Survey. These points, therefore, were located by angles from the triangulation points, which enabled the co-ordinates

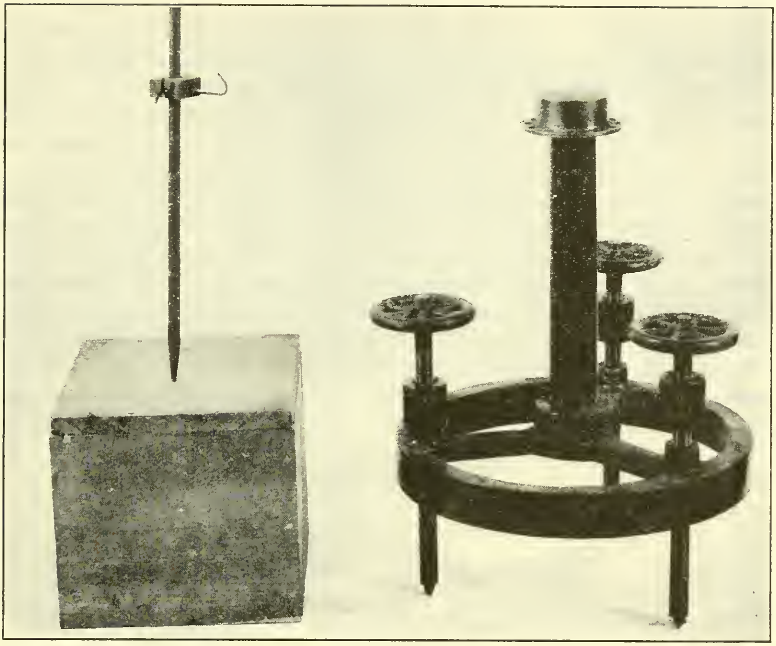


FIG. 4.—PICKET USED FOR INTERMEDIATE SUPPORT OF TAPE; AND SPIDER USED IN MEASURING BASE LINES.

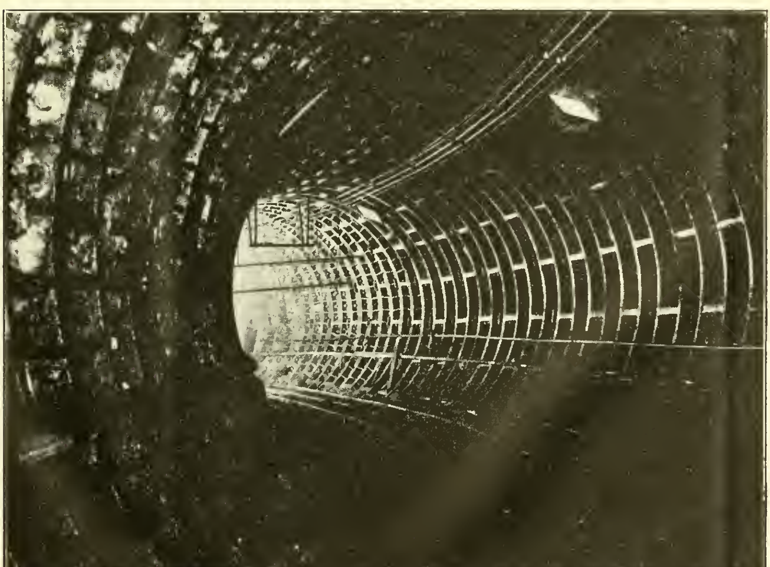


FIG. 5.—TUNNEL ON CURVE, WITH SUPPORTS FOR TRANSIT AND OBSERVER HUNG FROM ROOF.

of these points and points on the State line to be determined, and from which intersections with the tunnels were computed. In like manner, the reference points of the harbor bulkhead and pierhead lines were located, so that their points of intersection with the tunnel could be determined. Mr. Snyder.

The Hudson and Manhattan Railroad Company contemplates building in the future another pair of tubes across the river, and has already built short lengths of these tunnels on Cortlandt and Fulton Streets and at points where these tunnels cross under the Fulton Street tube, and certain junction enlargements on the New Jersey side. When this work is undertaken it will be necessary to drive the tunnels from new shafts, the position of which will then be determined, probably on the New Jersey side, and the objective will be these short lengths of tunnels and junctions already completed. It will then be necessary to determine the position of these new shafts with reference to the com-

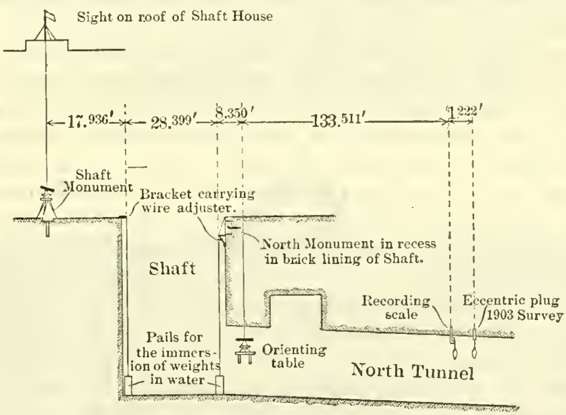


Fig. 6.

pleted work. It was felt that it would not be safe to rely on the monuments of the triangulation, as they are likely to be disturbed, and that more permanent points should be established on property within the control of the Company, as a basis for this future work. Therefore, two precise points within view of each other have been established and their co-ordinates determined, one being on the main power-house and the other on the elevator shaft-house of the Pavonia Avenue (Erie) Station in Jersey City (Plate IV).

The lines were transferred underground in the ordinary way by plumb-lines at the shafts (Fig. 6). This was made somewhat difficult on account of the small size of some of the shafts and the fact that some of them were placed to one side of the tunnels. The Morton Street shaft (New York City) was only 7 ft. in diameter, Mr. Snyder. and was placed between the tunnels, so that the direction of the line transferred was at right angles to the tunnels instead of parallel to them. The Pier *C* shaft (Jersey City) was located more than 400 ft. south of the nearest tube. At Cortlandt and Fulton Streets, New York City, the shields were started in chambers sunk in the form of caissons from the surface, and the lines were transferred by the use of two plummet wires lowered from the surface through pipes extending through the roof. In all these cases where the plumb wires were close, the position of the surveys was checked by plumb-lines in pipes driven from the surface 200 ft. or more from the initial point, so as to get the advantage of a longer base. Alignment scales, similar to those described by Mr. Noble, were used in ranging in the line at the shafts, but not elsewhere. The position of the line underground was the average of a large number of observations; at the 15th Street shaft (Jersey City) the first angle from the underground base was the mean of more than 400 observations. Lines were transferred underground at twelve points.

In the underground surveys two general methods were followed: the preliminary and the final. The preliminary lines were kept close to the face, for the proper guidance of the workmen. Owing to the movement of the iron lining for some distance back of the shields, due to the thrust of the rams and other causes, it was necessary to follow up and keep correcting this preliminary work with more precise surveys carried forward from points in the portion of the tunnel which had come to rest. In driving the shield through the silt without excavating, the tendency of the iron lining was to rise immediately in the rear of the shield and then gradually to come to rest. The maximum rise was about 1 in. at a point about 30 ft. in rear of the shield, and the iron would then gradually fall 3 or 4 in. below its original position. A shield being driven alongside a tunnel previously built would cause a slight lateral movement in the latter.

In the preliminary method all distances were measured at least twice, the tape being given a uniform pull of 12 lb., with a spring balance and without intermediate supports. Angles were repeated six times.

The final measurements in the underground work were carried on by methods similar to those used in the base line measurements on the surface, excepting that the spiders were not used, measurement being made to points plumbed down from the roof. The tape was supported on blocks at intervals of 20 ft., and the pull was obtained by a spring balance.

The method of prolonging or extending the lines differed from that used by Mr. Noble, it being to set a point ahead approximately on line and then to obtain its position by measurement of the distance and the exterior angles from the points in the rear. Similarly, in extending the

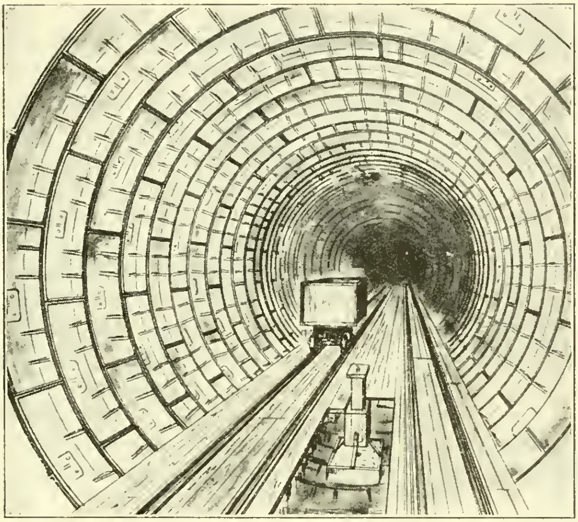


Fig. 7.—Telescoping Station Mark in Floor of Tunnel.

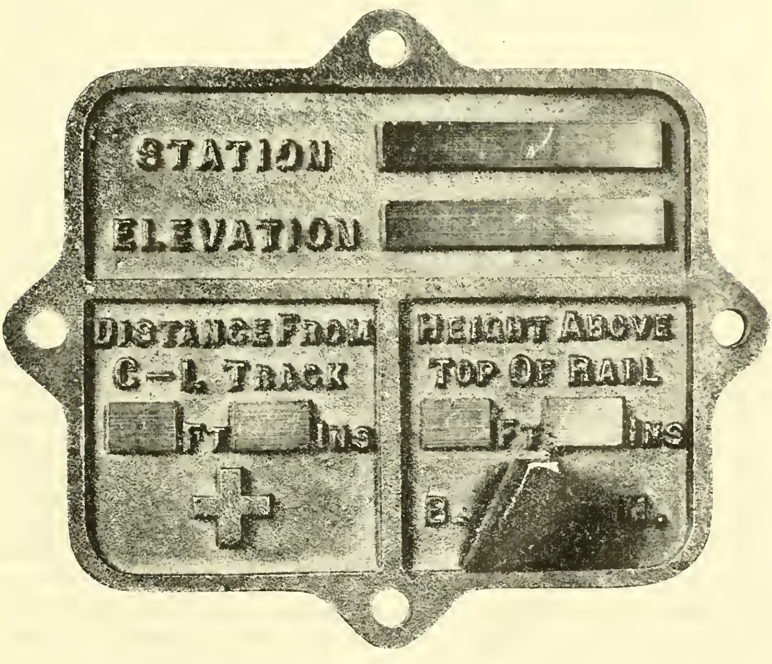


Fig. 8.—Reference Plates Set in Side of Tunnel, for Use in Lining and Surfacing.

lines through a lock, the projection of a straight line was not attempted, but a point in the lock was occupied by the instrument, and the angle to points on either side was obtained by repetition. Mr. Snyder.

The underground points were marked by small lag-bolts having flattened heads and a pointed hole so that a plumb-line would always take the same position. These were driven into strips of wood clamped to the tunnel bolts (Fig. 9).

Very important points, or where it was necessary to set points on a definite line, were marked by eccentric plugs (Fig. 9). These plugs, with sockets, were set approximately in line, and fastened rigidly to the tunnel structure. A fine hole for attaching a plumb-line was drilled near the rim of the head of the plug. The plug was then turned until the plummet hung exactly in line, and was then fixed in position by the jam nut.

Plummet lamps were not used in the underground work, but sights were taken to plummet lines which were made visible by holding in the rear a light screened by a frame of tracing cloth.

Some of the triangulation angles were turned at night, the sight rods being $1\frac{1}{4}$ in. in diameter and placed in front of 400-c.p. lamps, screened by a 30 by 40-in. sheet of tracing cloth.

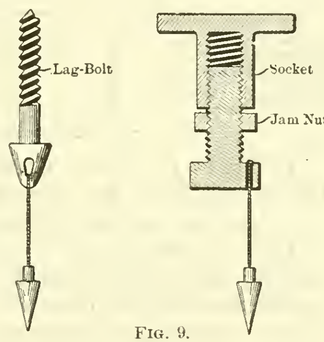


Fig. 9.

The method of suspending the transit and observer from the tunnel roof is indicated in Fig. 5. An adjustable standard for the support of the transit for use at points which had to be used frequently, and which was telescoped to below the construction platform when out of use, is indicated in Fig. 7.

For guidance in driving the shield, the construction force was given somewhat more information than on Mr. Noble's work. This consisted of: First, the position of the center of the last ring with reference to the theoretical axis of the tunnel, both in plan and elevation; that is, its position to right or left of the true center line and above or below the true grade line; second, the position of the face of the last ring referred to a plane normal to the theoretical axis of the tunnel; and third, similar information fixing the positions of the shield. In addition, the diameter of the tunnel was measured horizontally, vertically, and diagonally at angles of 45° from the vertical, in order to detect any tendency to distortion. This information was posted on a board at the face and also reported to the construction office on a prepared blank.

Mr. Snyder.

The position of the tunnel was usually checked daily at noon, but when unusual progress was being made, it was also checked at midnight, and on sharp curves sometimes every ring was checked.

The original leveling across the river was done by the use of tide gauges, but after the completion of the north tunnel, the levels were corrected by running through this tunnel.

Table 1 gives closing errors of a few of the longest portions of tunnel driven.

TABLE 1.—CLOSING ERRORS OF LONGER PORTIONS OF HUDSON AND MANHATTAN RAILROAD TUNNELS.

Shaft.	Depth, in feet.	Distance between plumb-lines, in feet.	Length of initial under-ground base line, in feet.	Longest heading driven from shaft, in feet.	Closing error in line, in feet.	Closing error in grade, in feet.
Morton St...........	65	5.4	205.7	4 524.1	No check.	No check.
Fifteenth St........	54	27.4	115.8	5 703.7	0.262	0.028
Washington St......	53.8	34.3	58.5	3 379.0	0.025	0.073
Pier *C*...............	83	277.4	261.0	5 240.0	0.164	0.004
Terminal...........	20	31.7	31.7	1 056.8		

The lines run from Morton Street to Twelfth Street and Sixth Avenue could not be checked, as the surface points had been disturbed at the time the shields were holed through.

Table 1 gives the results on only a few of the longer lines. In all, 23 headings were holed through in prosecuting the work. Some of the tunnels were shield-driven, with an iron lining, and some concrete-lined without a shield, and 19 shields were required. The tunnel built with each shield is indicated on Plate IV. With the exception of the old north tunnel, the maximum closing error was $3\frac{1}{8}$ in. for line and $\frac{7}{8}$ in. for grade, but most of the errors of grade have been less than $\frac{3}{8}$ in.

After the completion of the tunnels it was necessary to readjust the line and grade of the track on account of irregularities in driving, and reference plates were set in the sides of the tunnel for use in lining and surfacing (Fig. 8).

The original re-surveys of the old tunnel work were made by B. F. Cresson, Jr., M. Am. Soc. C. E. On the commencement of the construction work, the surveys were in charge of Ernest Statham, Resident Engineer. The surveys for the holing through of the old north tunnel, the first to cross the river, were made by the late E. Elbert Young, Assoc. M. Am. Soc. C. E., Engineer of Alignment. When the work was expanded and required a greater force, it was under the direction of two Division Engineers: F. K. Hilt and A. R. Archer, Assoc. M. Am. Soc. C. E.

The writer is indebted to the above, as much of the information in this discussion has been derived from their notes and records.

Mr. Ridgway.

Robert Ridgway, M. Am. Soc. C. E.—Mr. Noble has treated his subject in such a comprehensive way as to leave little to be said in discussion. The speaker has some knowledge of the work described, having been connected with it in its earlier stages, and regards it as the most difficult piece of tunnel surveying with which he is familiar.

The shafts on the opposite sides of the river were not visible from each other, the line of sight between them being obstructed by trees and structures, and the use of auxiliary alignment points was required. Two horizontal curves, one about 2 000 ft. in length, made it necessary to establish seven or more angle points in the tunnel. On the Brooklyn side the tunnel was in sand and silt, and the tubes shifted slightly for some time after the line points were established in them, making repeated checkings necessary in order to avoid serious error. In addition, the driving of the tunnel through the sand caused more or less surface disturbance, which affected the position of the alignment points on top, making it necessary to refer them carefully to offsets some distance back in the cross streets.

There was also the difficulty of passing the lines and levels through the air locks. To accomplish the successful results described by the author, in spite of these trying conditions, called for the patient application of sound common-sense methods. One does not realize from a reading of Mr. Noble's concise paper how much patient work was required to overcome the difficulties.

It is gratifying to know that the satisfactory results were obtained without the use of special instruments or appliances, only the ordinary surveying equipment of a tunnel construction party having been used.

Mr. Cresson.

B. F. Cresson, Jr., M. Am. Soc. C. E.—This paper is exceedingly interesting and valuable as a description of work done. It seems unfortunate that, notwithstanding all the surveys for tunnels and bridges which have been made in the vicinity of New York, very little has been written as to field methods and calculations, or the results obtained in the actual work. If a more thorough description of methods and calculations were to be written, it would be of benefit to those having charge of similar surveys in the future.

The speaker, as Alignment Engineer on the North River Division of the Pennsylvania Tunnel work, had charge of the surveys, triangulations, and calculations extending from the east side of Ninth Avenue to the portal at the west side of Bergen Hill.

Mr. Noble refers to measuring his base lines with a short tape and plumb-bobs. The method used in the Pennsylvania survey was quite different, and was devised with the idea of eliminating almost entirely the use of the plumb-bob for this purpose, as it appears that this instrument is not usually capable of securing great accuracy in

Mr. Cresson.

results. The speaker used a 100-ft. steel tape, the coefficient of which had been determined. This tape was supported every 20 ft., under a tension of 12 lb., attained by a weight operating over a wheel, and measurements were taken between movable station points placed usually about 99 ft. apart, the tape just touching the brass tops of the movable station points, on which were scratched fine cross-marks. The temperature was taken during each measurement, and the inclination of the tape was determined by a level. All work was done at night so as to avoid interference by traffic and obtain more uniform temperature conditions. By this method the use of the plumb-bob was eliminated, the measurements being taken by direct contact with the top of the movable station point, and being carried from station point to station point.

The monuments at the ends and along the base lines were tied in by sighting a transit (set up at right angles to the base line at the monument) on the point of the monument and reading on the tape. No effort was made to keep the tape level, elevations being taken at both ends to determine the slope. The results obtained by this method appeared to be very satisfactory. Different tapes were used on different nights, and a comparison was made between the results obtained on one base line on one side of the river and the other on the other side of the river, to aid in determining the coefficient of the tape.

In the river quadrilateral, the lengths of the base lines were, respectively, 2 263 ft. + and 2 242 ft. +, and the distance between the base lines was 6 688 ft. +. The smallest of the angles was 18° 41′ +, which was somewhat smaller than desired. The angles were turned with a 7-in., 10″ transit, which was set up, not by plumb-bobs, but by sighting other instruments on the monument and on the transits themselves. This was particularly necessary at the triangulation tower in Weehawken, which was 60 ft. high, as there was constant movement at the top of the tower.

The angles were read by a number of observers, and the whole system was carefully balanced, not only with respect to the angles, but also including the measured base line.

The levels were transferred across the river by prolonging a level base alternately from each side of the river, and the averaged results corrected themselves for the earth's curvature and for refraction.

The method of laying out parallel lines was to use a beam compass 37 ft. long—the distance between tunnel centers—and, from one tunnel line, to set off the parallel line. Excellent results were secured in this way, as the wooden beam was not affected by temperature or by tension.

There were many other details of these surveys and of the calculations which the speaker believes would be of interest if presented.

FIG. 10.—BASE LINE MEASUREMENT. PENNSYLVANIA RAILROAD TUNNELS.

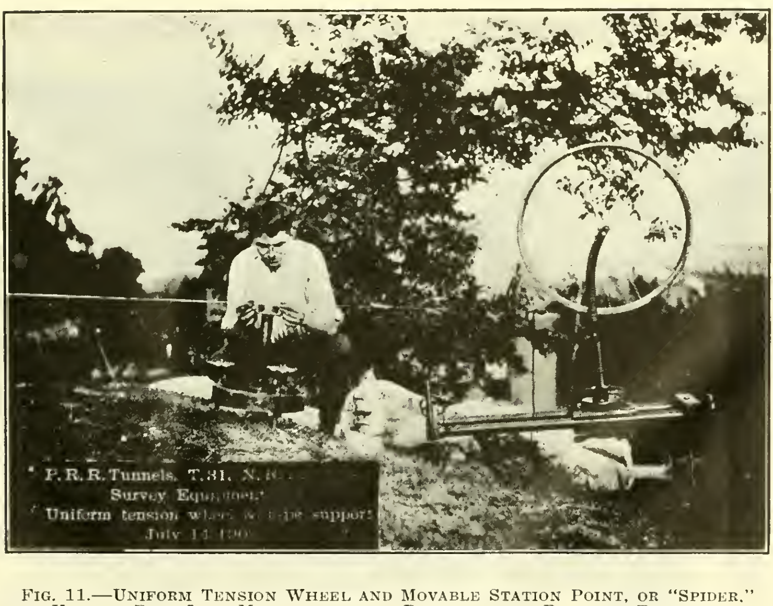


FIG. 11.—UNIFORM TENSION WHEEL AND MOVABLE STATION POINT, OR "SPIDER." USED IN BASE LINE MEASUREMENTS. PENNSYLVANIA RAILROAD TUNNELS.

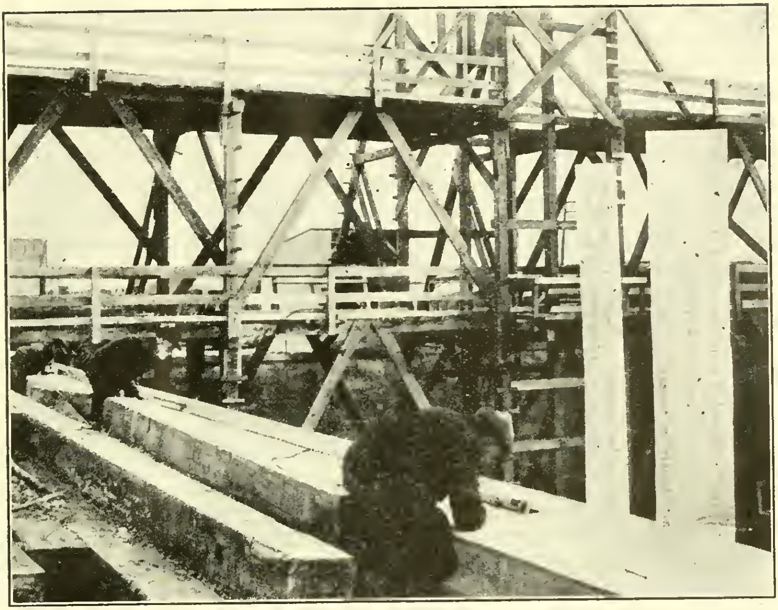


Fig. 12.—Beam Compass, 37 Feet Long, Set on Base, North Wall, Weehawken Shaft. Pennsylvania Railroad Tunnels.

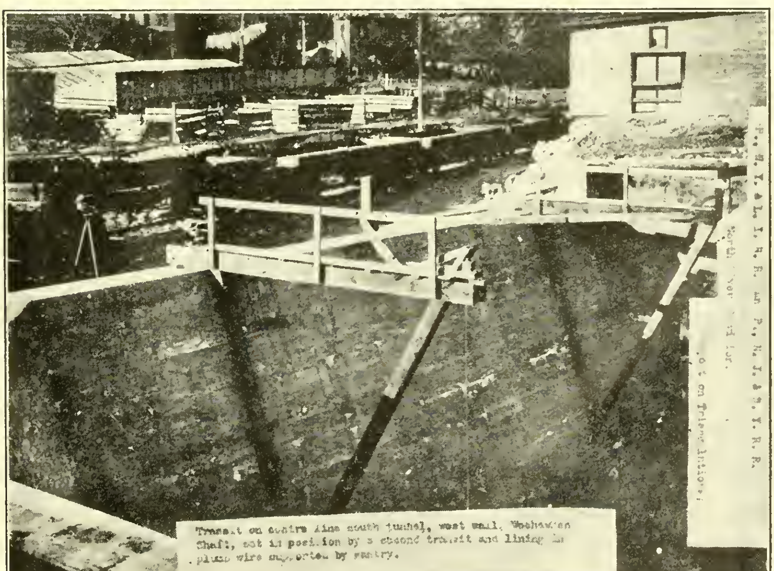


Fig. 13.—Transit on Center Line, South Tunnel, West Wall, Weehawken Shaft, Set in Position by a Second Transit, and Lining in Plumb-Wire Supported by Gantry.

S. M. PURDY, M. AM. SOC. C. E.—This paper is of great interest to engineers who are engaged in making surveys for important works, and will prove helpful to many. The literature on this subject is so meager, that many devices and methods which have been found highly successful by engineers in certain localities are unknown to their brethren, who have not been so fortunate as to observe their use. As an exposition of tunnel survey methods, triangulations, and base line work under most trying conditions, this timely paper will no doubt arouse considerable interest and lead to valuable discussion. Mr. Purdy.

In base line work, or in measuring tangents, where accuracy is required, the writer has found that the observation of a few simple principles has uniformly led, not only to excellent results, but to a great saving in time and labor.

Stakes or points are set on line with transit and tape in the usual manner, the stakes being driven at intervals somewhat less than the length of the tape to be used in the final measurement. This is in recognition of the fact that an unknown distance can be measured with a greater degree of accuracy than a known distance can be laid off. Also, if unknown distances are used, they can be checked with greater certainty, the personal element entering to a less degree, and the tendency to repeat errors being almost entirely eliminated. The writer has usually placed the stakes about 99½ ft. apart for a 100-ft. tape, and a corresponding distance for other lengths of tape. Tacks (usually small brass brads) are driven arbitrarily in the stakes for definitive points. The differences of elevation between adjacent stakes are next determined by a line of levels and are used in maintaining the level of the tape. Where possible it is desirable to hold the zero end of the tape fixed on one point, thus obviating the necessity of using two plumb-bobs. Thermometer readings are taken and recorded with each measurement.

During the construction of the Torresdale Conduit, in Philadelphia, the writer was called on to do work which was similar to that described by Mr. Noble. This conduit is a pressure tunnel, 10 ft. 6 in. in diameter, after lining, and is about 14 000 ft. long. It was driven through the rock at an elevation averaging 100 ft. below the surface of the ground, and the headings were reached through nine temporary and two permanent shafts. In transferring the alignment from the surface to the tunnel, the following method was used: A wooden stringer was placed across the top of the shaft, parallel to the center line and about 1 in. therefrom. To this stringer were fastened hangers, similar to those described by Mr. Noble, to which were suspended soft iron wires of No. 18 gauge, the latter sustaining weights (usually a piece of scrap iron) of about 30 lb. The distance apart of these wires, forming the length of the base line, was determined by

Mr. Purdy. the conditions of the shaft, but in all cases was as great as possible, varying from 3.2 ft. in Shaft No. 7, to 10.4 ft. in Shaft No. 4. It was not generally found necessary to place the weights in water, as it was thought that a small oscillation of the plumb wires was preferable to absolute steadiness.

A transitman, whose instrument was set up a short distance from the shaft, kept the tops of the plumb wires in alignment by constantly testing them, checking his own position at frequent intervals.

Two transits were used in the tunnel. They were set up at either side of the shaft, about 15 ft. from the nearest wire. These transits were equipped with a special device by which a lateral motion could be attained with a slow-motion screw. Each transitman proceeded to align his instrument with the two wires, observing first one and then the other. When a transitman announced that he was on line, he was required to set a stake about 100 ft. away from his instrument, using his foresight. Subsequently, the second transitman set a similar stake. Observer No. 1 then plunged his telescope and tested the point set by Observer No. 2, and Observer No. 2 sighted on the point set by Observer No. 1. In this manner a base line some 200 ft. long was established in the tunnel, which was immediately referenced to permanent points. A week or two later this entire proceeding was repeated, and if the two lines failed to agree, a third test was made. This was found to be a rapid, convenient, and accurate method for transferring lines, and the work was easily performed during the noon hour, when tunnel excavating was not in progress, thus causing practically no inconvenience or delay to the contractor.

Lines were carried into the headings as the work progressed. Points for line and grade consisted of horse-shoe nails through the heads of which were drilled holes $\frac{3}{16}$ in. in diameter. These nails were driven into wooden plugs which in turn were driven into drill holes in the roof of the tunnel. Plumb-lines were suspended from the nails whenever it was desirable to use a point. For excavating purposes, a piece of blasting wire was fastened to the nail and to this was tied a small stone. When not in use the wire was coiled up out of the way.

During 1898 a large topographical survey was made by the United States Deep Waterways Commission. Incidental to this survey, a triangulation was made of Oneida Lake, a body of water about 20 miles long, and from 2 to 6 miles wide. For this work an ordinary transit with verniers reading to 20″ was used. A requirement of the Commission was that quadrilaterals should close within 10″. At first, there was some difficulty in meeting this requirement, and the experiment was made of reading the angles at night, using lamps as stations. This proved highly successful, and night observations were continued until the completion of the work.

The whole triangulation was made during the winter while the

Mr. Purdy.

lake was frozen. Both the starting and closing base lines were measured from shore points across intervening ice. Standard tapes, 100 ft. long, were used, lying flat on the ice, the proper tension being applied by a spring balance.

The method of repetitions was used in reading the angles. The party consisted of an observer, two vernier readers, and a recorder. A complete set of readings for any angle consisted of six sets of six repetitions each. The vernier was first set at zero and pointings were taken from left to right, both verniers being read each time. After reading the angle, then twice the angle, and so on for six pointings, the observations were taken from right to left until the plates were brought back to zero. Failure to check as close as 20″ caused the rejection of the entire set. For the next set, the vernier was set at 60°, and readings were taken in a similar manner. For the third set the verniers were set at 180°, and so on. By changing the set of the vernier each time, the whole limb of the transit was brought into use, and any errors of eccentricity which may have existed were eliminated.

The writer is of the opinion that observations taken at night would be of material assistance in triangulations in and about a large city. Certainly the atmospheric conditions at night are more equable, and as there is less traffic there is less vibration and also better opportunities for rapid work. Again, a flame affords several advantages as a point on which to sight, freedom from phase being perhaps the most important.

Mr. White

Lazarus White, Assoc. M. Am. Soc. C. E.—The speaker was connected for a time with the work described by Mr. Noble, took some part in the surveys, and is glad that their accuracy was verified so well by the actual meeting of the headings. He is now connected with tunneling which demanded a great deal of preliminary work of the kind described by Mr. Noble. It is probable that, due to the numerous shafts through which the City Aqueduct Tunnel is to bring Catskill water into New York City, the amount of alignment work necessary is unprecedented.

The twenty-four shafts are located, for the most part, in parks and at the intersections of city streets, and between them runs the tunnel. The most difficult part of the work, of which the writer is in charge, is that which is to be built under Contract 67. In order to avoid the condemnation of private property for rights of way, the tunnel is to be built under narrow streets on the lower East Side, crossing the East River to Brooklyn at the foot of Clinton Street. As an illustration: there are five angles in the tunnel between Shafts 20 and 21, both of these shafts being more than 700 ft. deep, and located on offsets to the main tunnel. Owing to the narrowness of the streets, there is a clearance of only a few feet at the corners.

Mr. White. The speaker does not believe that elaborate apparatus is now necessary for accurate base-line traversing, or for triangulation in connection with tunnel alignment. Most of the taping on the City Aqueduct was made with the simple apparatus shown by Fig. 14. In addition to the usual equipment of levels, tape, sight-rods, etc., it was necessary to have only two tables, constructed of 4-in. pipe, and an outfit consisting of turnbuckle, spring balance, and stretcher, as shown. This additional apparatus cost only a few dollars. The tapes were standardized according to the methods shown on Fig. 16, and with the apparatus of the Topographical Bureau of The Bronx. This apparatus is all that can be desired, and with it the tape can be readily standardized to the utmost degree of accuracy.

Fig. 16 also shows the tables used, and a copy of the actual field notes for one run. These tables are very simple, and were prepared for the purpose of eliminating the error in the application of temperature corrections, each tape length being calculated for its actual distance at certain temperatures, with suitable pull and catenary. The tables also contain a correction in case the tape is supported. After adding up the tape lengths, it was necessary to make only slope corrections, obtained from the level runs. These slope corrections are always in one direction, and, therefore, cannot be applied with the wrong sign.

Before taping, angle-points of traverses were established in the streets. Where the traffic was light, the points were taken directly on the sidewalk and used with an elevated target, shown on Fig. 14. This target proved to be convenient, as it could be supported securely above the heads of pedestrians. In places where the traffic was heavy, or for long lines, elevated points on buildings or other structures were selected and keel-marked, and angle-points were obtained by successive trials between the elevated points. These elevated sights proved to be of great help; intermediate points on the line were established by running toward them, and, in addition, the angle work was done with speed and was not obstructed by traffic. Angles were measured by repeating six times with telescope direct, reading the first, fourth, and sixth readings; then measuring the supplementary angle in the same way, with telescope reversed, the averages from the first, fourth, and sixth readings checking within from 3 to 5″, and the sum of the angle and supplement being 360° within 5″. When the results exceeded these limits, additional work was done, and both the angle and its supplement were measured twelve times—six direct and six reverse.

Taping was done with the apparatus shown, the sequence of operations being as follows:

(1) Rear turnbuckle adjusts index of tape to pin on table;

(2) Stretcher puts on slight tension, aligning tape over keel-marks on sidewalk, or sighting along line;

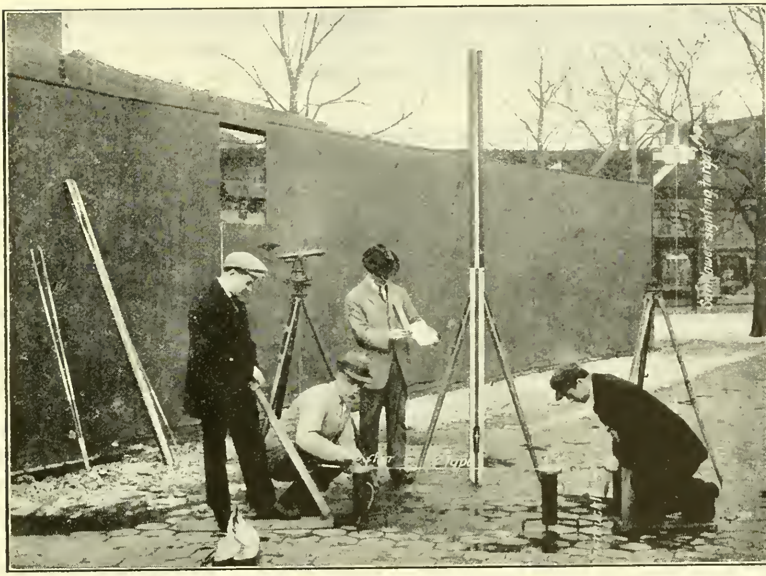


FIG. 14.—APPARATUS USED FOR BASE-LINE TAPING, FOR CITY TUNNEL, CATSKILL AQUEDUCT.

FIG. 15.—APPARATUS USED FOR BASE-LINE TAPING, RONDOUT PRESSURE TUNNEL.

Mr. White.

(3) Head chainman meanwhile places tripod under tape to solid bearing, marks keel triangle and keel station on the ground;
(4) Stretcher puts on the proper tension, and pin is stuck opposite index;
(5) Tension is released, rear turnbuckle is unhooked, fore chainman goes ahead, sliding tape on ground, rear chainman brings his box and table (A) with him to the stretcher, who has been guarding the forward table (B); the stretcher then goes ahead with table (A) to his new station, and operation is repeated.

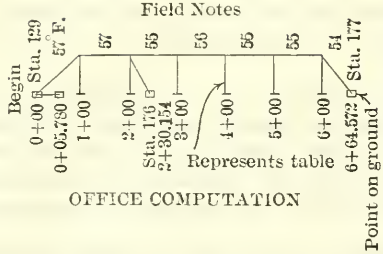
After the taping was completed, a double line of levels was run over the triangles obtained during Step (3). The note-keeper recorded the temperatures and kept graphic notes, showing exactly how the tape was stretched along the ground. These graphic notes are reproduced, in part, on Fig. 16, and were of great help to the levelmen and in the computations, making it very easy to apply the proper corrections. Forward and backward runs readily checked one another in the field, as the slopes were the same and corrections for average differences of temperature could be readily made.

All the shafts were connected by closed traverses, as indicated on Fig. 17, from which it will be seen that the errors of closure in these traverses were very small. In addition, an entirely independent check was obtained by tying in the Smith-Gray Tower, in Brooklyn, and the Metropolitan Life Tower, in Manhattan, the co-ordinates of both these points being established by Government traverses. These two points, about 20 000 ft. apart, checked within 1 ft., although the co-ordinates of the Metropolitan Life Tower are not guaranteed by the Government to be correct within 1 in 25 000. The traverses as originally run out were not generally over the center line of the tunnel, but from them the exact positions of the P. I.'s of the tangents vertically over the tunnel were computed. Between these P. I.'s distances were taped and angles turned, as a final check on the computations in the original field work. These were found to check closely with the computed lengths and angles.

The East River was crossed by the triangulation system shown on Fig. 17, only a few days' work being necessary. The Water Street base line was first taped, the Brooklyn base line being computed and found to check within 0.08 ft. when afterward taped. The angles were read by repetition in a set of ten direct and then a set of ten reverse; the first, fourth, seventh, and tenth angles of each set being read and averaged, to insure against slipping plates and wrong readings. The angles were read on March 13th and 14th, 1911, under fair conditions. In every case the signals sighted at were solid points—parts of the building structure—except at Corlear Park, where a range pole was supported by a tripod. There were two main triangles with

Mr. White.

FORWARD RUN
Field Notes



OFFICE COMPUTATION

Sta. 129 to 177

Point	Diff. Elev.	Slope Corr.	Temp.	Distance
129 0+00				
175 0+05.780	0.20	.004		05.780
1+00	0.48	.001	57	100.010
2+00	0.96	.005	57	100.011
176 2+30.154	0.39	.003		30.157
3+00	1.06	.006	55	100.009
4+00	0.62	.002	56	100.010
5+00	0.80	.003	55	100.009
6+00	0.60	.002	55	100.009
177 6+64.572	1.69	.023	54	64.578
		.042		664.636
			less	.042
		129 to 177 = 664.594		
		129 to 175 = 05.776		
		175 to 176 = 224.393		
		176 to 177 = 434.425		

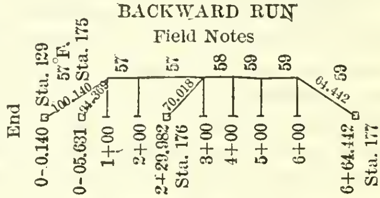
Intermediate plus measurements data shown thus: (0.004) Slope corrections are taken from printed table.

TAPE LENGTH TABLE
Tape unsupported between ends

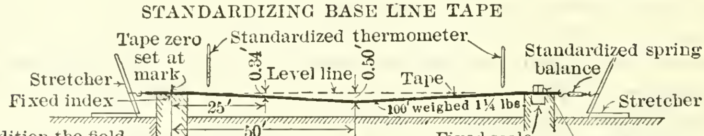
Temp. Fahrenheit	Tape No.6			30 lbs pull
	0-25	0-50	0-75	0-100
0	24.993	49.988	74.981	99.973
20	24.997	49.994	74.991	99.986
30	24.998	49.997	74.996	99.993
32	24.999	49.998	74.997	99.994
34	24.999	49.999	74.998	99.995
36	24.999	49.999	74.999	99.997
38	25.000	50.000	75.000	99.998
40	25.000	50.001	75.001	99.999
At two degrees intervals up to 90°				
86	25.007	50.016	75.023	100.029
88	25.008	50.016	75.024	100.031
90	25.008	50.017	75.025	100.032
100	25.010	50.020	75.030	100.038
For Tape supported add	.000	.001	.003	.007

For intermediate distances the correction is proportional.

BACKWARD RUN
Field Notes



STANDARDIZING BASE LINE TAPE



In addition the field thermometers and balances are compared with the standard.

First: Standard tape is stretched and scale read
Second: Tested " " " " " "

Vernier is set by screw motion opposite tape index and scale read through microscope to $\frac{1}{10\,000}$ ft.

From the above, the supported length was computed. Then accurate ¼ tape lengths were laid out on level floor inside a building and the errors at 25, 50 and 75-foot graduations were found for tape supported and unsupported. The sag correction results were later closely checked by formula; Sag= $\frac{Lw^2 l^2}{24 t^2}$

BASE LINE TAPPING
CITY TUNNEL
(CONTRACT 67)
CATSKILL AQUEDUCT
NEW YORK
BOARD OF WATER SUPPLY.

Fig. 16.

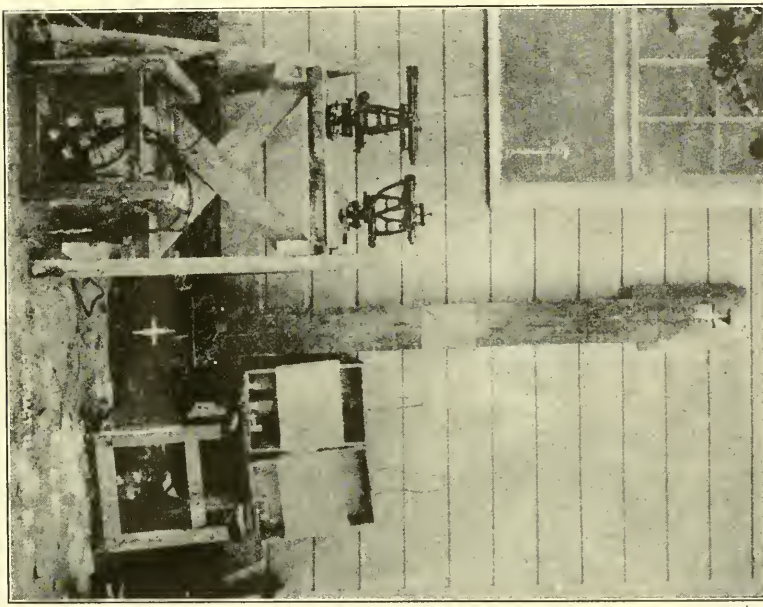


FIG. 18.—SIGHTING BOARD AND APPARATUS USED FOR LINE DROPPING, RONDOUT PRESSURE TUNNEL.

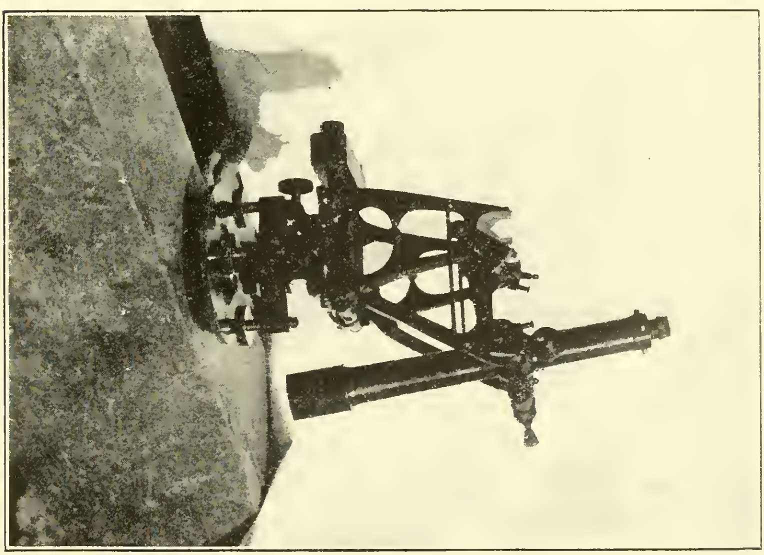
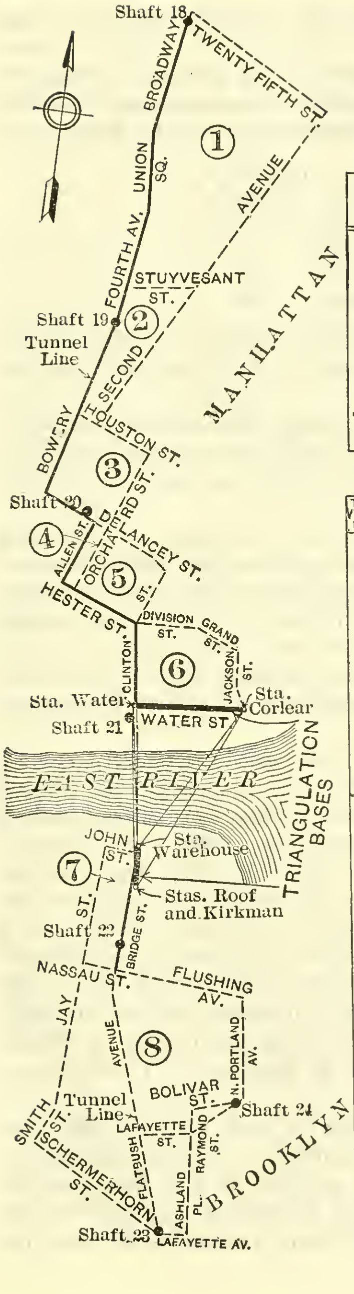


FIG. 19.—VERTICAL SIGHTING TRANSIT USED FOR LINE DROPPING, RONDOUT PRESSURE TUNNEL.

Mr. White.



BASE LINE TRAVERSING
CITY TUNNEL
(CONTRACT 67)
CATSKILL AQUEDUCT
NEW YORK BOARD OF WATER SUPPLY.

ERROR OF CLOSURE IN TRAVERSES

TRAVERSE NUMBER	NUMBER OF ANGLES	ERROR	LENGTH TRAVERSE MILES	ERROR OF CLOSURE
1	10	0°00′20″	2.4	1:51 000
2	8	0°00′11″	1.2	1:42 000
3	4	0°00′10″	1.1	1:53 000
4 & 5	10	0°00′12″	1.0	1:47 000
6	7	0°00′04″	1.1	1:105 000
7	6	0°00′01″	1.0	1:85 000
8	11	0°00′20″	2.6	1:190 000
7 & 8	15	0°00′19″	3.4	1:395 000
Triangulation	Brooklyn base, computed from Manhattan base, differs 0.083 foot from measured distance.			

TAPING NOTES
Some of the Shorter Courses Omitted.

TRAVERSE NO.	STATION		DISTANCE TAPED			
	FROM	TO	FORWARD	BACKWARD	THIRD RUN	AVERAGE
1	M.7	M.4	2 293.655	2 293.658		2 293.656
	M.4	M.1	1 198.023	1 198.003		1 198.013
	160	40N	1 220.503	1 220.541	1 220.610	1 220.551
	M.12	M.9	1 801.164	1 801.173		1 801.168
	M.0	M.8	860.825	860.799		860.812
	M.8	M.7	595.462	595.472		595.467
2	198	195	762.321	762.289		762.305
	195	196	328.311	328.312		328.311
	196	143	937.583	937.592		937.589
	150	M.1	420.809	420.825		420.817
	199	198	766.836	766.852		766.844
3	192	175	1 480.431	1 480.459		1 480.445
	129	133	357.189	357.193		357.191
	133	137	837.481	837.503		837.492
	137	196	1 486.777	1 486.761		1 486.768
	195	192	1 014.542	1 014.541		1 014.542
4	119	125	228.380	228.377		228.378
	125	126	501.907	501.918		501.912
	126	127	366.442	366.431		366.436
	127	128	256.034	256.046		256.040
	128	129	189.654	189.653		189.654
5	175	177	658.818	658.843		658.831
	177	212	545.804	545.805		545.804
	212	210	985.584	985.583		985.584
6	Corlear	Water	1 881.799	1 881.796	1 881.815	1 881.803
	107	110	692.772	692.770		692.771
	114	171	993.950	993.937		993.944
	171	173	798.080	798.084		798.082
	173	174	618.975	619.032	619.004	619.002
	174	Corlear	338.908	368.894		368.901
7	32	35A	460.764	460.764		460.764
	35A	46	1 657.111	1 657.119		1 657.115
	46	60	546.539	546.534		546.536
	60	61	1 294.971	1 294.940		1 294.956
	61	64	818.494	818.506		818.500
	64	32	530.446	530.441		530.444
8	64	67	1 842.689	1 842.691		1 842.690
	70	72	574.344	574.328		574.336
	72	77	2 502.743	2 502.876		2 502.810
	77	6	595.018	595.992	595.025	595.012
	6	7	441.300	441.327		441.314
	7	10	1 642.121	1 642.154		1 642.138
	10	11	601.916	601.927		601.922
	11	15	325.933	325.946		325.940
	15	19	1 729.240	1 729.229		1 729.234
	19	64	2 808.619	2 808.563		2 808.591

FIG. 17.

Mr. White. a common base in Manhattan to locate two points in Brooklyn at the extremities of the Brooklyn base.

A definite idea of the time taken by the work indicated on Fig. 17 is shown by the following: (The field work was done between March 1st and May 14th, 1911, including the triangulation across the East River.)

Base-line taping done	=	118 000 ft.
Base-line traverse run	=	65 000 ft.

Number of 7-hour days:

Worked on taping	=	12
Worked on traversing	=	42

The time spent on base-line traverse includes all miscellaneous work, such as locating building corners, leveling, secondary traversing, etc.

It is to be noted that this work included the accurate location of building lines and the intersections of all streets under which the tunnel is to be built, this information being necessary in order to determine the clearances.

The lines to be taped were laid out by transit ahead of the taping by putting marks on the sidewalk at approximately 50-ft. intervals, or, when there was a good foresight, the line was sighted in, either by the fore- or rear-chainman, and was checked at line-points existing at every block. This method leads to little error when it is considered that a 100-ft. tape length must be offset 0.45 ft. before the distance is in error 0.001 ft., and offset 0.63 ft. before the distance is in error 0.002 ft., or 1 in 50 000.

With the method just described, the working speed, including delays, was very little less than 2 500 ft. an hour, even on crowded streets. Through the use of the elevated sights, as described, crowds did not hinder the aligning. A 100-ft. tape length was regularly taped in 90 sec., including the time taken to move ahead to get ready for the next length, and a speed of 4 500 ft. an hour has been made. The essentials leading to speed are the short time (from 2 to 5 sec.) that the tape is actually stretched to make the measurement, and the sliding of the tape along the ground in moving ahead. On Sunday, April 23d, 1911, the party taped and ranged out the line from Orchard Street, along Delancey Street, along the Bowery to Fourth Avenue, to 15th Street, and return—a distance of 14 500 ft.—in 6¼ hours.

The principal source of error in taping with ordinary steel tape such as used, however carefully standardized, is due to the temperature of the tape not corresponding to that of the thermometers. This source of error can be practically eliminated with the "Invar" tape, which is made of an alloy of iron having a coefficient of expansion only one-twenty-eighth of that of steel. These tapes were originally made in Europe, but are now made in the United States, and cost about $1 per ft. with graduations every foot for 50- to 200-ft. lengths. They are well worth the increased cost where much accurate taping is required. The "Invar" tapes are soft, however, and require large reels and careful handling. Mr. White.

The method of taping described is essentially the same as that used up the State on base-line taping in connection with the work of the Board of Water Supply, with the exception that, on account of the rough ground, high tripods were used. The tapes were hand-stretched, without the use of stretcher or turnbuckle. This is shown on Fig. 15. Here, also, the tape lengths were always 100 ft., the data for slope corrections being obtained by simultaneous leveling on top of the tripods, the temperature of the tape being observed at the same time. With this apparatus, high speed was also made, a distance as great as 5 000 ft. being taped in both directions, over rather rough country, in one day. With this method, to-and-fro taping was computed in the field, so that it was at once known whether a third taping would be necessary.

In dropping lines down the shafts, the apparatus and methods used were very similar to those described by Mr. Noble. The shafts were much deeper, however, being as great as 700 ft., and even 1 100 ft. at the Hudson siphon. In this work it was found that the most important feature is to perfect the mechanical details, so that when the party goes out the apparatus will be in readiness for quick work. At first the shafts were provided with alignment boxes, through which wires for line-dropping were suspended. These proved to be a source of considerable worry, as one could never tell whether or not the line was touching the side of the box. Later, they were removed, as it was found better to drop the wires in the open. The chief source of error in work of this kind is the likelihood of the fouling of the wires. It was found that it was not necessary to occupy the shafts more than two or three Sundays in order to obtain lines sufficiently accurate to run between shafts 4 000 or 5 000 ft. apart and meeting within less than a few tenths of a foot. This was appreciated by the contractors, as the engineers interfered with the shaft only a few days in the course of the work.

On the Rondout and Wallkill Siphons a vertical sighting instrument was used with considerable success. This instrument was devised to drop the line directly down the shaft by sighting with a telescope arranged to be mounted on a bracket so as to clear the plates when in a vertical position. The instrument was built by Berger, from parts used in previous models, and was excellent for the purpose, except that it was subject to lateral movement on the wyes,

Mr. White.

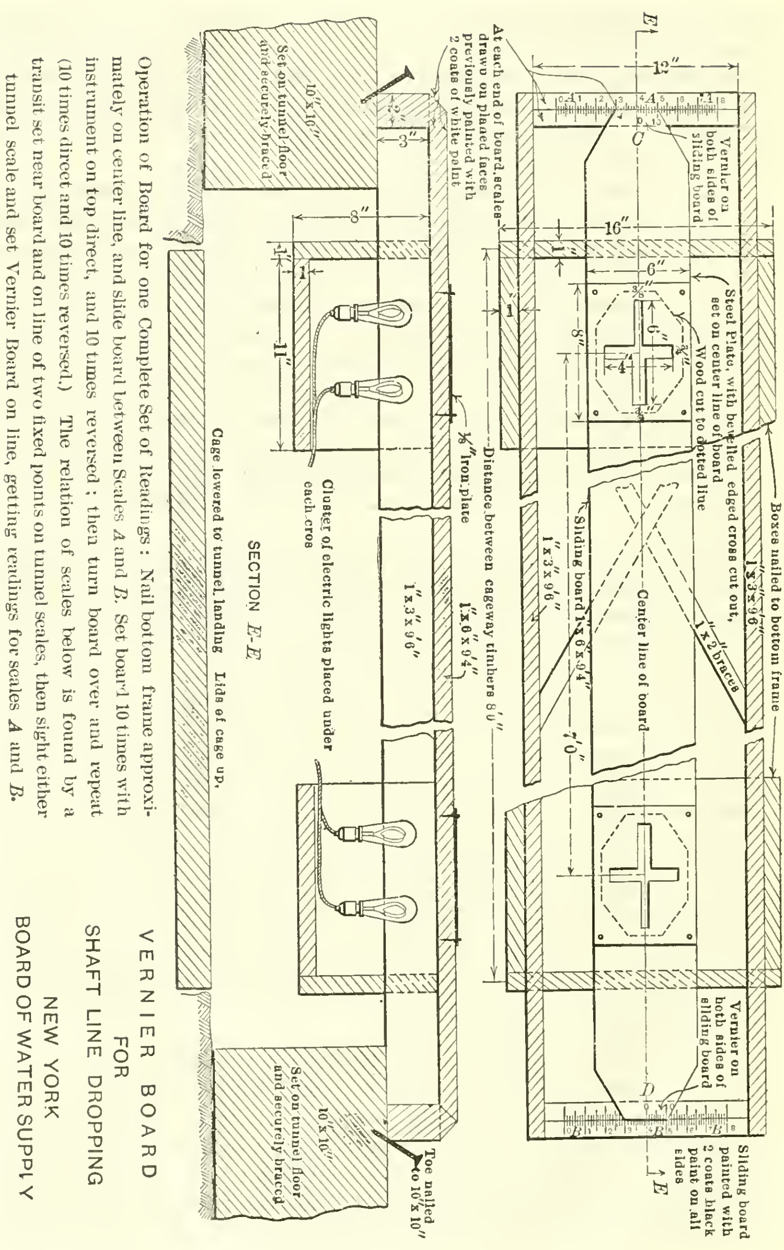


Fig. 20.

there being no positive adjustments to prevent this. This instrument permitted a method entirely different from that generally used, and provided an independent check sufficiently good to justify running out lines with only two sets of observations—one obtained by line-dropping with wires, and the other with the vertical sighting instrument. The method used with this instrument is shown clearly by Fig. 20. It was found that the results obtained depend very largely on the mechanical details, lighting, ventilation of shaft, etc. Fig. 18, shows the sighting board and apparatus for dropping line down the shaft by vertical sighting, as used on the Rondout Tunnel. Fig. 19, shows the vertical sighting transit. Mr. White.

Frederick C. Noble, M. Am. Soc. C. E. (by letter).—The method of measuring base lines described in the discussion is admittedly more accurate than the one described in the paper, yet the latter gave very satisfactory results. Since the tunnel line had curves between shafts, the triangulation base was measured to the principal end that the headings should meet with the minimum error in line; and, to this same end, equal consideration had to be given the tunnel measurements carried forward from the shafts. Compensating errors, such as inaccuracies in individual measurements resulting from the use of ordinary apparatus, are to be distinguished from systematic errors, which do not thus balance out. Known systematic errors can be allowed for, but there will be systematic errors of unknown extent peculiar to the method. Provided the same method of measuring is used in the tunnel as for the base line, the sum of the systematic errors peculiar to the method should be of the same sign, and proportional to the distances measured, thus leaving the final result unaffected. The best method of measuring a triangulation base for tunnel purposes should be a simple one which proves to be the most readily applied to measuring in the tunnel. Mr. Noble.

In all measuring and alignment operations a large number of repetitions is the best way of reducing the effect of unavoidable individual errors. How far this should be carried in any case depends on a proper recognition of the importance of the final result as compared with the importance of the means taken to secure it.

Co-ordinates were used for computation purposes in special cases where conditions warranted, but there was no occasion to make their use a general feature of the survey work. All topography was readily referable to the principal lines from which the working tunnel lines were derived.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1220

THE HALLIGAN DAM: A REINFORCED MASONRY STRUCTURE.*

By G. N. Houston, M. Am. Soc. C. E.

With Discussion by Messrs. Maurice G. Parsons, Lars R. Jorgensen, S. G. Swigart, Charles B. Buerger, Maurice C. Couchot, L. J. Mensch, Edward L. Sayers, and G. N. Houston.

The Halligan Dam is on the North Branch of the Cache La Poudre River, about 12 miles above the Town of Livermore, 35 miles northwest of Ft. Collins, Colo., and 16 miles from the nearest railroad siding. It impounds the water of that stream in a reservoir having a capacity of 6 408 acre-ft., being one of sixteen which, with their connecting canals, form the Irrigation System owned by the North Poudre Irrigation Company, of Ft. Collins. This is a stock company capitalized for $400 000, practically all the stock being held by the owners of the land on which the water is used.

Before the writer was retained as Consulting Engineer, plans had been prepared for an arched cyclopean masonry dam, with a gravity section, to store water to a depth of 55.5 ft. above the outlet tube, and a contract had been let for its construction at cost plus 20 per cent. Work was commenced, but, after building to a point slightly above the lower outlet tube, the available funds were exhausted and the work was temporarily abandoned. The foundations remained in this condition for about 18 months, and then the work was resumed under the writer.

* Presented at the meeting of December 20th, 1911.

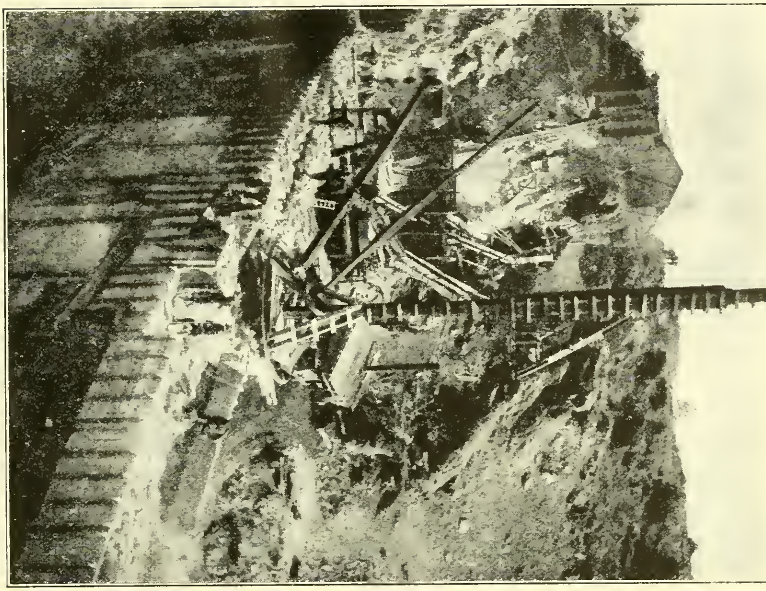


FIG. 1.—UP-STREAM FACE, DURING CONSTRUCTION.

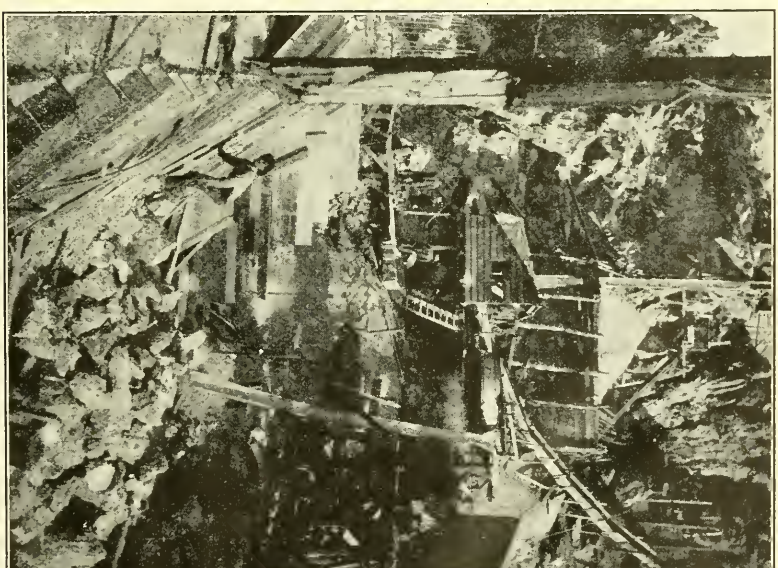


FIG. 2.—DOWN-STREAM FACE, DURING CONSTRUCTION.

The problem was to obtain as much storage capacity as possible for a limited sum of money, and the question then became how far to depart from established precedent in order to meet the economic conditions. There was the gravity section, built in the form of the arch, as the ultra conservative type, and the Bear Valley Dam as the extreme in the other direction. The writer's design is submitted as one which best fulfills the requirements, and one in which the stresses can be demonstrated to be safe.

New plans and specifications were prepared for a cyclopean masonry dam, reinforced with steel rods, to be built on the foundations already constructed. This dam is of lighter cross-section than the original gravity type designed, and 14.3 ft. higher.

This additional height increased the storage capacity by at least 2 500 acre-ft., which, at the usual price of $40 per acre-foot, is worth $100 000. A conservative estimate shows that the dam was built at a cost of $7 000 less than if it had been constructed according to the original plan. In other words, the company saved $107 000 by the use of this design instead of the usual gravity section.

There are about 3 500 cu. yd. of masonry in the old work, and 12 134 cu. yd. in the new structure, making a total of about 15 634 cu. yd. in the whole dam. The cost of the old work was more than $100 000, that of the new work was about $80 000. Below Elevation 0 the cost was $6 per cu. yd., and above that elevation it was $6.25. Work was begun under the writer's supervision in July, 1909, and completed on May 1st, 1910. The contractor was the C. G. Sheely Contracting Company, of Denver.

Construction.—The foundations, as left by the first contractor, were very uneven, and, before beginning the new structure, it was necessary to use about 1 400 cu. yd. of masonry to bring them up to the elevation of zero, as shown by Fig. 3.

For 35 ft. above this point the dam was constructed of cyclopean masonry, consisting of large, irregular masses of rock, varying in size from 1 cu. ft. to 2 cu. yd., bedded in a 1:3:5 concrete, mixed wet. Smaller rocks were used to fill in between the large masses, 6 in. being the minimum distance between the stones. The quantity of cement used in the concrete for this part of the work was about 1.20 bbl. per cu. yd. The rock content averaged 27% of the total mass.

After the construction had reached approximately Elevation 35,

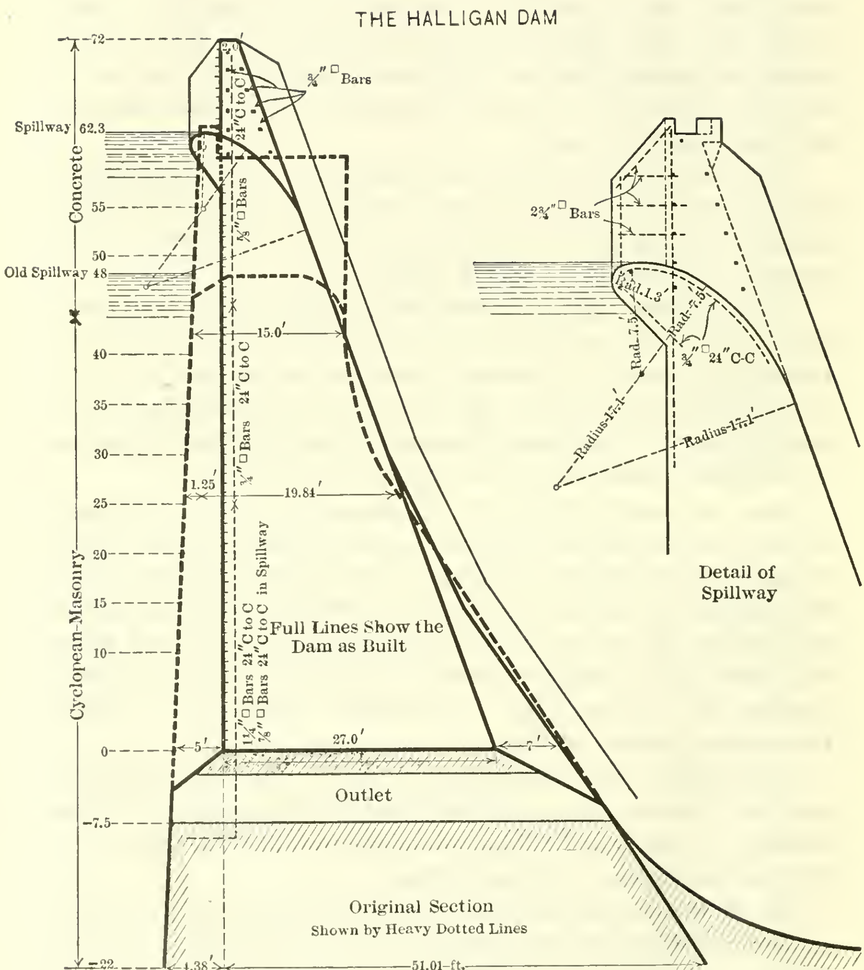

THE HALLIGAN DAM
-72
Spillway 62.3
55
50
Old Spillway 48
40
35
30
25
20
15
10
0
-7.5
-22
Concrete
Cyclopean-Masonry
¾″ □ Bars
21″ C to C
⅝″ □ Bars
15.0′
¾″ □ Bars 21″ C to C
1.25′
19.84′
1¼″ □ Bars 21″ C to C
⅞″ □ Bars 21″ C to C in Spillway
Full Lines Show the Dam as Built
5′
27.0′
7′
Outlet
Original Section
Shown by Heavy Dotted Lines
4.38′
51.01–ft.
2¾″ □ Bars
Rad. 1.3′
Rad. 7.5′
Rad. 7.5′
¾″ □ 24″ C-C
Radius-17.1′
Radius-17.1′
Detail of Spillway

Fig. 3.

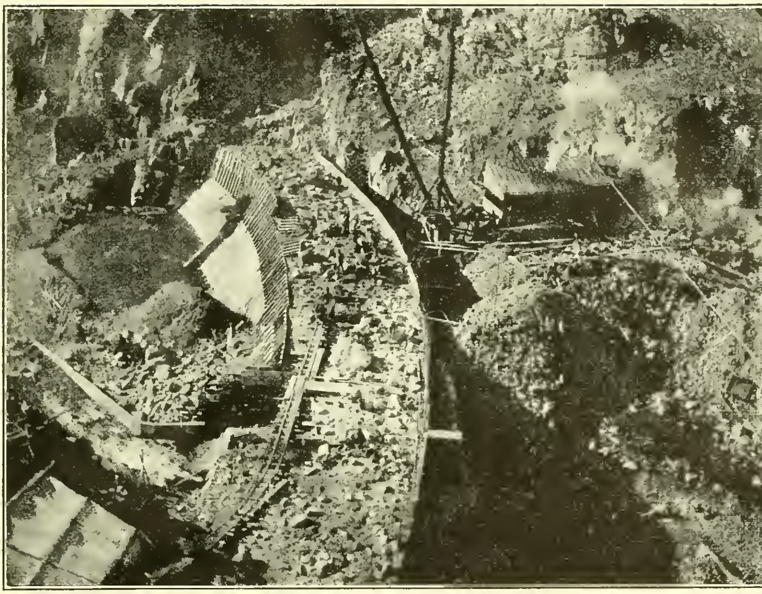


Fig. 4.—View from Above, During Construction.

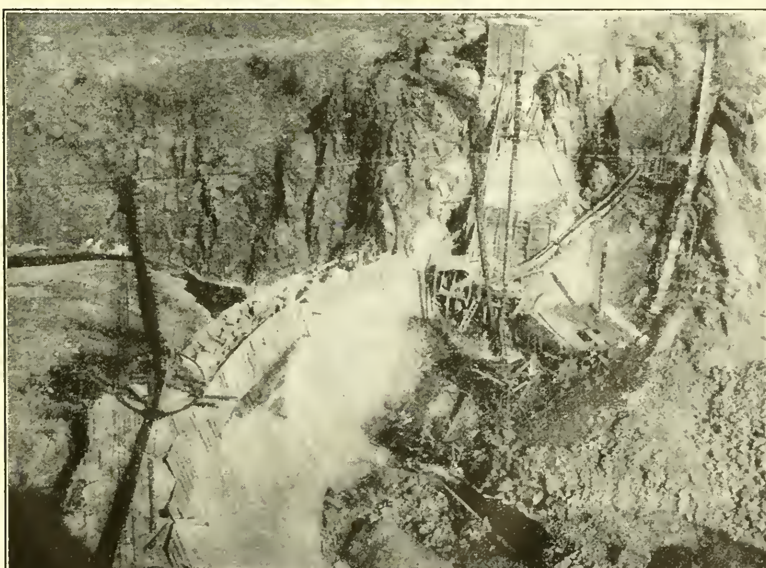


Fig. 5.—View from Above, During Construction.

the character of the large rock became so poor that it was decided to build the remainder of the dam entirely of 1 : 3 : 6 concrete. The projecting lip of the spillway, however, was built of a mixture of about 1 : 2½ : 4. The crushed rock varied from a small percentage of pieces having a greatest dimension of 6 in. to pieces ½ in. in diameter. All material finer than ½ in. was excluded on account of the large percentage of mica which it contained. The sand used was very coarse, although it varied somewhat. An average of nine analyses gave the following result:

Size.	Percentage, by weight.
Larger than ¼ in. in diameter, classed as gravel...	8.8%
Passed ¼-in. mesh..................................	91.2%
" No. 20 sieve, 0.034 in. in diameter......	29.2%
" No. 30 " 0.022 " " "	15.2%
" No. 100 " 0.0045 " " "	2.2%

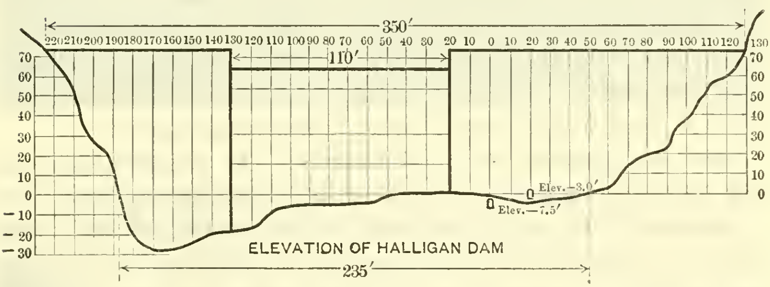


FIG. 6.

All the cement was furnished by the Colorado Portland Cement Company, and, with the exception of about 1 000 bbl., was tested at the George Pierce Testing Laboratories, at Portland, Colo.

TABLE 1.—SUMMARY OF TESTS OF 10 000 BARRELS OF IDEAL PORTLAND CEMENT USED IN THE CONSTRUCTION OF THE HALLIGAN DAM.

	Tensile Strength.					Fineness.		Time of Setting.			
	Neat.			One to three.		Passed 100 mesh.	Passed 200 mesh.	Initial set.		Final set.	
	1 day.	7 days.	28 days.	7 days.	28 days.			H.	Min.	H.	Min.
Maximum.....	480	945	970	415	490	98.8%	87.7%	5	20	9	55
Minimum.....	150	625	700	235	315	94.0%	74.2%	2	15	3	55
Average.......	311	706	828	327	400	96.3%	79.2%	4	0	6	36

Concrete was laid during the day through all but the severest winter weather. The water was heated, and varied in temperature from 130 to 160° Fahr. An attempt was made to heat the sand, but was abandoned as not necessary. The temperature of the concrete as deposited in the dam varied from 35 to 60° Fahr. Before bedding the large rocks they were cleaned with a jet of steam. During the night the new work was covered with tarpaulins, under which lighted lanterns were placed. Concrete was not mixed when the temperature of the atmosphere was lower than 20° Fahr. These precautions were sufficient to prevent serious freezing of the concrete.

The bulk of the reinforcement was of high-carbon steel. Two samples of this steel were tested at the Laboratories of the University of Colorado, and showed an elastic limit of 50 000 and 59 000 lb. per sq. in., respectively. A small quantity of twisted steel was also used in the upper part of the dam. Where the elevation of the old foundation was — 3, or lower, the reinforcement was bent, as shown on Fig. 3, and built into the work. Where the foundations were higher than this, holes 1 in. larger in diameter than the bars were drilled from 3 to 5 ft. deep, and the bars were set in neat cement grout.

The dam presents two novel features: The projecting spillway, and the light cross-section of both the main dam and spillway.

Spillway.—The writer originally contemplated a spillway wholly within the cross-section of the dam, but his attention was called to the fact that a discharge over it having a depth of from 2 to 3 ft. would probably leave the face of the dam. As there was a possibility of discharges of much greater depth, it was necessary to widen the crest of the spillway in some way, hence the projecting lip. The surface of the spillway is a compound curve, formed by the arcs of three circles, and following as nearly as possible the curve of discharge of an 8-ft. head of water over a sharp-edged weir, as determined by Bazin's experiments. Reinforcement was placed in the lip, as shown by Fig. 3, and needs no special comment. The capacity of the spillway is about 14 000 sec-ft., and that of the two outlet tubes is about 1 400 sec-ft.

Stresses.—The following analysis of the stresses in the dam is based on the following assumptions:

First: That the high-water level is 9.7 ft. above the spillway, that is, to the top of the dam.

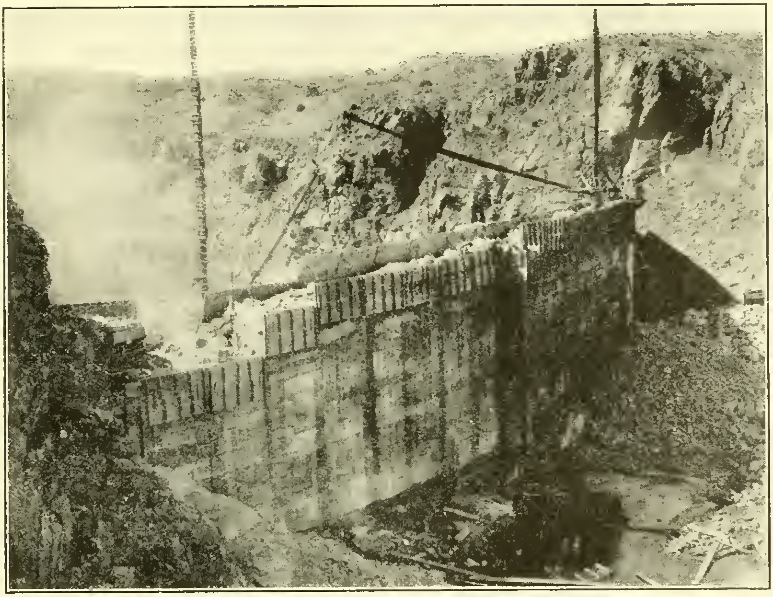


FIG. 7.—CONCRETE FORMS IN PLACE.

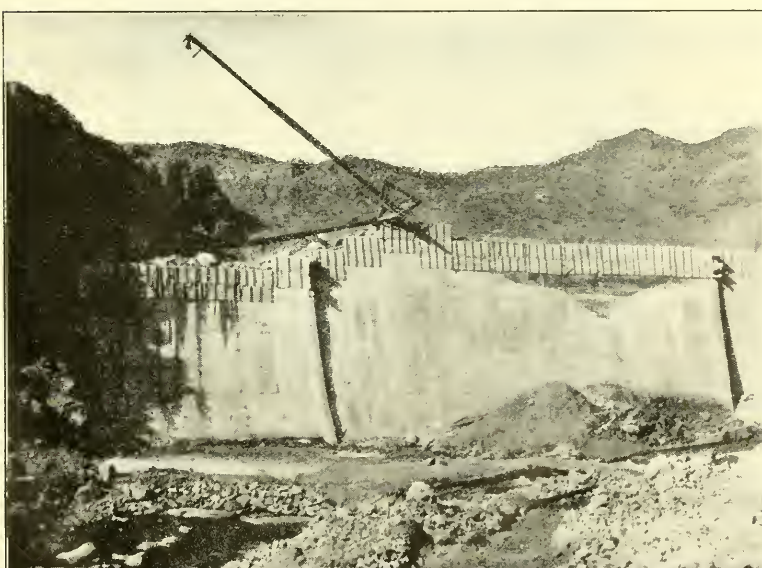


FIG. 8.—DOWN-STREAM FACE, SHOWING SPILLWAY.

Second: That the dam resists the water pressure partly as a cantilever beam and partly as an arch.

Third: That the center of gravity of the compressive forces is at a point distant from the down-stream face 0.15 of the effective thickness of the dam.

Fourth: That the dam section is a complete triangle, thus making the perpendicular line through the center of gravity of the section cut the base at a point one-third the thickness of the dam from the rear face.

Fifth: That the safe load (tension) on the reinforcement is 12 000 lb. per sq. in.

Sixth: That the neutral axis of the dam, considered as a beam, is at a point 0.4 of the effective thickness from the down-stream face.

Seventh: That the thrust in the arch is uniformly distributed over the thickness of the dam.

The general method used is to assume the safe load on the reinforcement, find what part of the water pressure can be safely carried by the dam considered as a beam, and then assume that the arch carries the remainder of the pressure.

In the following equations the various functions are represented by the following letters:

P = Total overturning pressure on the dam, in tons;

P'' = That part of P assumed to be sustained by the beam action, in tons;

P' = That part of P assumed to be sustained by the arch action, in tons;

w = Weight of 1 cu. ft. of water, in pounds;

r = Radius of curvature of dam, in feet, = 324;

j = Thickness of the dam at any point, in feet;

h = Height of the dam at any point, in feet;

W = Weight of masonry in the dam above any point, in tons;

T = Thrust in arch due to P', in tons per square foot;

y = Assumed stress in steel, in tons;

c = Maximum compression in the outer face of the dam due to beam action alone, in tons per square foot;

z = Total compressive stress in dam due to beam action alone, in tons;

q = That part of the pressure at any point sustained by arch action alone, in tons.

Fig. 9 is a diagram of the stresses acting on the dam.

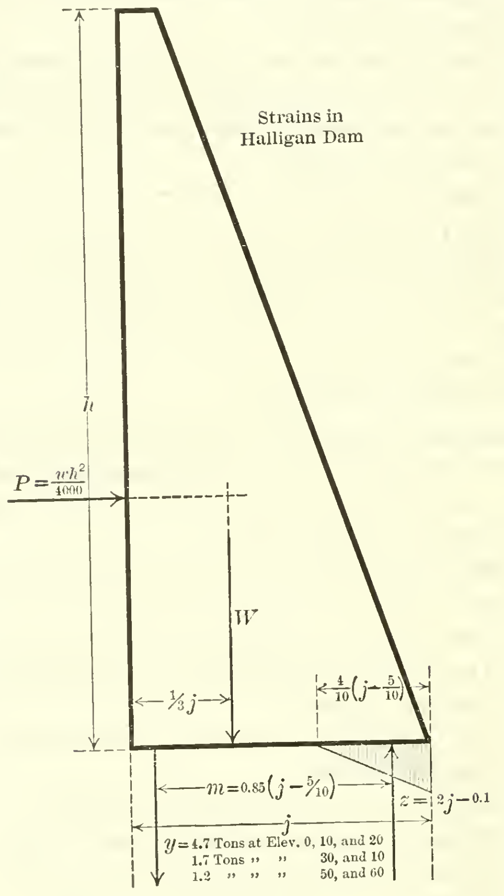


Fig. 9.

At Elevations 0, 10, and 20; y = 4.7 tons,
" " 30, and 40; y = 1.7 tons,
" " 50, and 60; y = 1.2 tons.

$$m = 0.85\ (j - 0.5)$$

$$P = \frac{wh^2}{4\,000}$$

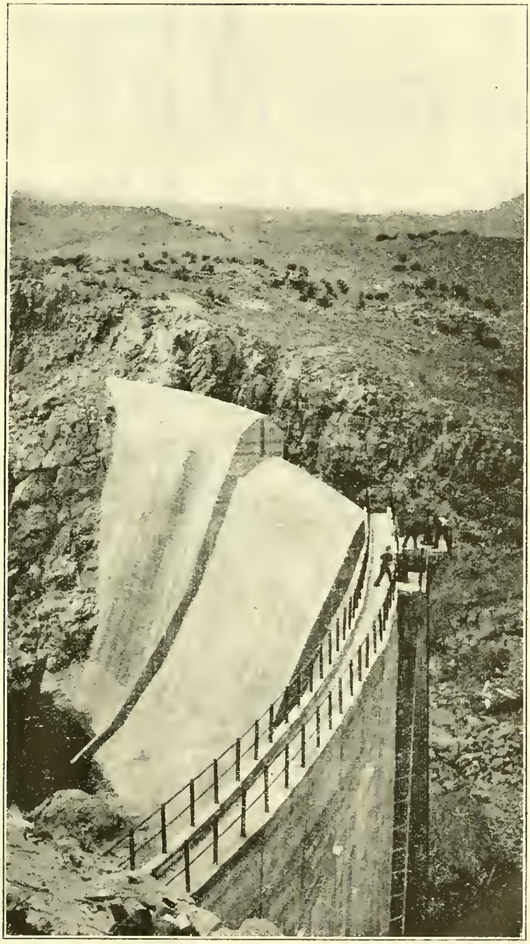


FIG. 10.—THE COMPLETED DAM.

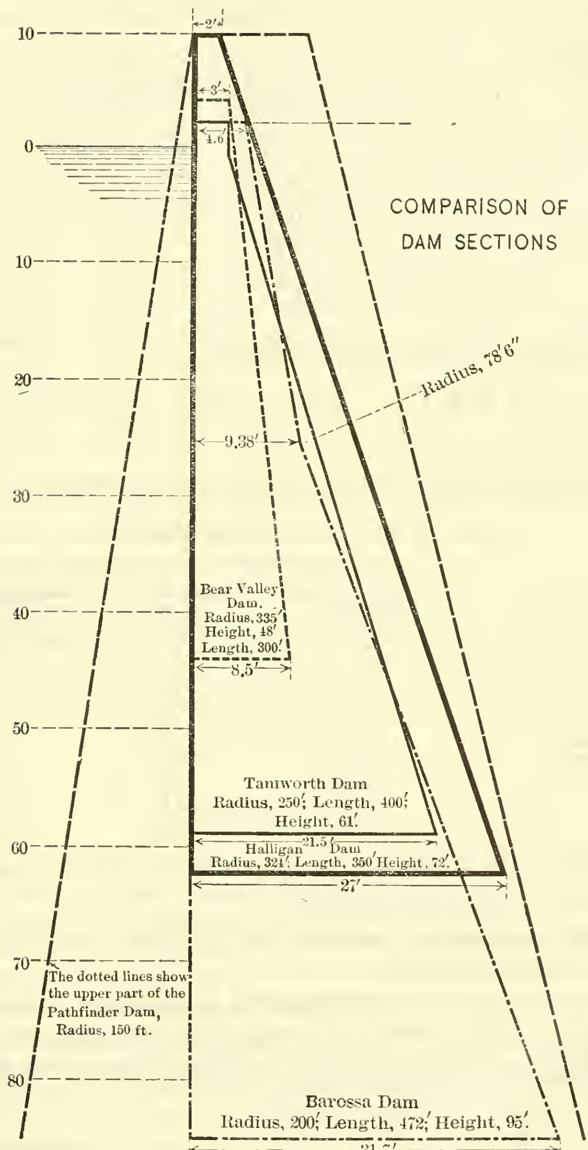

10
0
10
20
30
40
50
60
70
80
2′
3′
4.6′
COMPARISON OF
DAM SECTIONS
Radius, 78′ 6″
9.38′
Bear Valley
Dam.
Radius, 335′
Height, 48′
Length, 300′
8.5′
Tamworth Dam
Radius, 250′ Length, 400′
Height, 61′
21.5′
Halligan Dam
Radius, 324′ Length, 350′ Height, 72′
27′
The dotted lines show
the upper part of the
Pathfinder Dam,
Radius, 150 ft.
Barossa Dam
Radius, 200′ Length, 472′ Height, 95′
31.7′

FIG. 11.

Expressing moments around the center of gravity of compression forces, we get,

$$\frac{1}{3} P'' h - W \left(m + 0.5 - \frac{1}{3} j\right) - y\, m = 0$$

Solve for P''.

Then
$$P' = P - P''$$

$$q = \frac{2P'}{h}$$

$$T = \frac{q\, r}{j}$$

To determine c, take moments around y:

$$\frac{1}{3} P'' h + W \left(\frac{1}{3} j - 0.5\right) - c\,(0.2\, j - 0.1)\, m = 0$$

Solve for c.

The stresses obtained by this method are shown in Table 2.

TABLE 2.—SUMMARY OF STRESSES.

Elevation.	Tension in steel, in tons per square inch.	Thrust in arch, in tons per square foot. T.	Compression due to beam action alone, in tons per square foot. c.
62.3 Spillway	0.0	0.0	....
60	0.0	0.0	....
50 Calculated	4.5	0.0	5.8
40 Assumed	6.0	4.4	8.0
30 "	6.0	7.2	9.7
20 "	6.0	7.5	12.3
10 "	6.0	9.3	13.5
0 "	6.0	10.5	15.7
0 "	10.0	9.1	16.3

The factor of safety is about 6.

Fig. 11 is a diagram comparing the Halligan Dam with several others which are well known.

The writer wishes to acknowledge his indebtedness to Charles W. Comstock, M. Am. Soc. C. E., State Engineer of Colorado, for valuable suggestions given during the design and construction of this dam.

DISCUSSION

MAURICE G. PARSONS, JUN. AM. SOC. C. E. (by letter).—It is gratifying to see the arch dam gradually coming into favor, for, in the writer's opinion, the Profession has heretofore given in too easily to the views of the lay mind as to the stability of these structures. For generations engineers have successfully built arch bridges dimensioned by methods of analysis no more certain than those applied to arch dams. A satisfactory theory of investigating the stresses in bridges has but recently been developed, and dams designed by the arch formula can be investigated under combined arch and cantilever action. Mr. Parsons.

If properly designed and constructed, such dams can fail only by being undermined, overtopped, or actually squeezed to death, while, within certain radii, they effect an economy of material; are less subject to temperature and shrinkage cracks than straight dams; and horizontal joints in them are rather to be commended than condemned. Most certainly the Bear Valley and Sweetwater Dams—one the thinnest in the world, and the other proof by trial against severe spill shocks—should highly recommend the arch type. With the advent of concrete and the development of arches, the multiple-arch dam seems to be the ultimate masonry type.

In relation to the Halligan Dam, the arch formula, $T = r\ g$, gives a unit stress of 27 tons per sq. ft., or 375 lb. per sq. in., at Elevation 0, under the assumption of a 72-ft. head, which pressure is within the bearing power of good masonry. Developed from a consideration of the simultaneous deflection under arch and gravity action, we have:

$$S_a = \frac{h^2\ x^2\ S}{2\ r^2\ b^2 + h^2\ x^2} \cdots\cdots\cdots\cdots (1)$$

$$S_g = \frac{2\ r^2\ b^2\ S}{h^2\ x^2 + 2\ r^2\ b^2} \cdots\cdots\cdots\cdots (2)$$

When $S = 62.5$, S_a and S_g are the numerators in fractions having 62.5 as the denominator representing the proportion of the total external force carried by arch and gravity action, respectively. In these equations: $r =$ the radius, in feet; $h =$ the head, in feet, to the bottom, where the width is b, in feet; and $x =$ the distance, in feet, of the section in question above b. These formulas give a gravity action of 100% at the bottom. "Trimming," by reason of the stresses obtained from an investigation by these formulas, would not be good practice, the formulas being useful for investigation, but not for design. If provision is to be made for up-stream tension, the rational procedure is to determine its extent, and then design to meet it. Being developed on the supposition of a triangular section, the actual division of stress as given by these formulas is indeterminate in this case, and, accordingly, the author had to make assumptions. It is presum-

Mr. Parsons. able that he formulated a good guess, and has the proper quantity of steel, at his safe unit stress, to care for the tension resulting from the line of pressure, reservoir full, falling without the central third of the base. The formulas show the danger of assuming a low total stress in the steel; if the cantilever action be enough greater than that assumed, rupture of the steel and failure as a cantilever will result.

With the gravity dam there is also the question of cantilever action, for masonry is elastic, and experiments prove up-stream tension in designs supposed to obviate all but compressive stresses. As there are gravity and arch formulas, so a dam could be designed as a cantilever of uniform strength. Assuming a slice 1 ft. thick, its side elevation would be obtained as follows:

$\frac{S\,I}{c} = M = \frac{w\,h^3}{6\,d}$, in which $S =$ the allowable stress, $h =$ the head over the horizontal section, the section modulus of which is $\frac{I}{c}$, and $w =$ the weight of 1 cu. ft. of water. In the case of a homogeneous material and a rectangular plan, $\frac{I}{c}$ is $\frac{b^2\,d}{6}$. When $d = 1$ ft., as in this case, then, $S\,b^2 = w\,h^3$,

$$\text{and } b = \sqrt{\frac{w}{S}\,h^3} = k\,h\,\sqrt{h}.$$

A dam dimensioned by this, as by the arch formula, would have additional stability because of gravity.

In dealing with arches as generally constructed, there is not only arch, but also gravity and cantilever action. In spite of the fact that the arch theory is based on the assumption of rings which enable the dam, at any depth, to act as if an infinitesimal lamina existed, in practice, the thrust and resulting stresses at various depths must be integrated by the masonry. There is a question whether the use of steel, in hindering free laminal action, is justified.

Furthermore, considering the subject of steel in general, it may be remarked:

1. That the ancient metal upon which is based the belief that reinforcement will last forever, is of a different nature than the present steel.

2. Other structures are not subjected to dampness to such an extent as dams, and, therefore, the use of steel in them is more conservative.

3. Streams of Western America are often far from chemically pure, so that the corrosion of the steel used to hold them back may be expected.

4. Engineers have gone reinforcing mad.

In dealing with arch dams, it would seem best to go by the formula, $T = r\,g$. There may be additional stability for a time, because gravity and cantilever action will exist to a certain extent unless preventive measures be taken. After these actions give out, sole reliance devolves on the arch action, but the dam cannot then fail without being crushed. Some day horizontal cracks will form and the steel will give way, thereby destroying any cantilever action. Again, from its very nature, the arch should be constructed in laminæ. Then cantilever action will be impossible, or investigation thereof unnecessary, for separate arch rings can act independently, and internal stresses can be lessened. Mr. Parsons.

The author is to be congratulated on the result. His dam is safe as long as the steel lasts, and, after that is gone, if it ever is, the structure will still be safe, even with horizontal cracks. He has built a monument, bold in its conception and yet entirely substantial, while, according to his figures, it has cost less and has risen higher than the dam originally designed.

Lars R. Jorgensen, Assoc. M. Am. Soc. C. E. (by letter).—The author states that the dam presents two novel features: the projecting spillway and the light cross-section. Of these two features, the projecting spillway is the best, and is a good solution of the problem. The cross-section is not too light, but the radius is unnecessarily long. The factor of safety of a dam, where dependence is placed on arch action, as in this case, is proportional to the thickness of the dam divided by the length of the up-stream radius; that is, just as much depends on the length of the up-stream radius as on the thickness. Mr. Jorgensen.

With water to the top of the dam, 72 ft., the mean axial compression is practically 25 tons per sq. ft. of the cross-sectional area. While there are several dams in the design of which this stress has been used successfully, it is at present considered somewhat high, especially for a comparatively thin circular column like the Halligan Dam. If the Bear Valley Dam has a factor of safety of between 2 and 3, as it probably has, the Halligan Dam should have a factor of safety of 5, at least. With a dam, however, there is always uncertainty as to the penetration of the hydrostatic pressure into the body, and, therefore, the factor of safety should be extra high.

In the middle of the dam, cantilever action takes place, and the steel rods help the arch to carry the load, together with the compression, in a horizontal plane along the down-stream face. This cantilever action diminishes from a maximum at midstream to zero at or toward the abutments, for the reason that the deflection of the arch has its maximum value in the middle, between the abutments, and has no deflection at the abutments, because it is held there. If the arch does not deflect near the abutments, it does not give the vertical beam (the cantilever) an opportunity to help. The axial compression at and toward the abutments, therefore, will be very nearly equal to the

Mr. Jorgensen. full 25 tons per sq. ft., whereas, in the middle, the axial compression will be much less, because the steel rods in that neighborhood are given a chance to serve the useful purpose of holding down the up-stream face of the dam.

An up-stream radius of about 200 ft. would have produced a dam in which the material would have been better utilized. The most economical up-stream radius is equal to the width of the cañon divided by the constant, 1.84, and to this is added the distance from the center of gravity to the up-stream face. It is true that the longer arch requires more material, but the factor of safety is increased in a much greater proportion by using the shorter up-stream radius. Using the shorter up-stream radius, and the same quantity of materials (and therefore a thinner section), will give a stronger dam, as the material will be better placed.

Mr. Swigart. S. G. Swigart, Esq. (by letter).—The writer, who was the designer of the original gravity dam and had supervision of its construction up to the time when work was stopped owing to the financial difficulties of the irrigation company, wishes to make a few corrections.

Mr. Houston states that the cost of the work previous to his taking charge of it was more than \$100 000, and that about 3 500 cu. yd. of rubble concrete were in place. The fact is that the cost of the construction up to that time was just about \$92 000, and, by actual cross-sections, more than 6 000 cu. yd. of rubble stone concrete were in place.

The author fails to mention anything about the cost of the excavation, much of it in a very hard but seamy granite (at one point 43 ft. deep in rock), mixed with a red talc formation which could not be shot out, but had to be picked out by hand; the cost of the diversion of the river; the erection of the plant; the stripping of the quarries; and the fact that more than 2 000 cu. yd. of rock were quarried ready for crushing, about 300 cu. yd. crushed, and about 1 500 cu. yd. of sand hauled to the works, ready for use. Nor does he mention the fact that all lumber used for forms, scaffolding, falsework, buildings, camp, sand and cement chutes, crushed-rock bins, etc., and the cast-iron gates and raising devices were bought and paid for, hauled to the dam, and that one of these gates had been placed in position.

The excavation was very difficult and expensive. It was under water, having been classed as wet excavation, and cost more than \$26 000. The cost of the diversion of the river was about \$1 000; freighting the material to place \$4 500, and erecting the machinery (which was all left in place ready for use, and was used on the later work) \$500. The cost of material left on the ground was about \$12 000, and that of the gates and raising devices about \$1 600, making the amount to be deducted from the total previously given, about

Mr. Swigart.

\$45 500, leaving for the concrete work proper approximately \$46 500. Much of this work was 1:2:4 concrete, in such places as the cut-off wall under the heel of the dam, around the outlet tube, and 2 ft. thick over both faces of the entire dam. The bed-rock, which was very irregular, was first covered with a thick coat of 1:1 mortar. (This portion of the dam was all in the spillway section, thus being subject to heavy erosion on the down-stream face.)

The author may not have intended to convey a wrong impression, and his figures were probably based on incorrect information; but the writer feels that the statements herein corrected, would naturally give a false impression as to the actual cost of the work under his charge. Also, regarding the percentage contract, while the writer does not approve of that form of contract in general, the percentage (20%) was based entirely on the labor cost, the material, office, and engineering expenses being specifically excluded, leaving it as based on what was actually only 60% of the total cost; in other words, if figured on the total cost as a base, it was only 12 per cent.

The work was carried on by the contractors, The Walter Sharp Construction Company, represented by Messrs. Walter Sharp, the President, and J. P. Brackett, the Secretary-Treasurer, who were both on the ground and in charge of the work at all times, and conducted it in a faithful and honest manner. Considering all the difficulties encountered and the naturally greater expense of placing concrete in the foundation under a heavy head of water, as well as in the winter, with all the necessary additional precautions against frost, the work was handled with at least reasonable efficiency, and every bit of it was well done.

When the work was suspended for lack of funds, the contractors offered under bond to complete the remaining portion of the dam, which was simply a question of straight concrete work without further danger from floods, at a total cost of \$75 000 for the dam complete, up to 10 ft. above the original plan. If their offer had been accepted, the dam would have been completed two years before it was finally finished, and the irrigation company would have had the benefit of two years' storage of water.

As to the design of the reinforced structure built on the foundation and portion of the original dam, while the writer does not agree with Mr. Houston as to the matter of assuming that the arch will carry all the strain not taken by the reinforced concrete calculated as a vertical cantilever beam, and would prefer greatly, especially with an arch of this radius (324 ft.), to be on the conservative side and make no calculation on the arch carrying any strain whatever, or, in other words, simply adding that much to the factor of safety, he does not wish to enter into a discussion of the design.

Mr. Buerger.

CHARLES B. BUERGER, ASSOC. M. AM. SOC. C. E. (by letter).—Any determination of the stresses in a dam combining the features of arch, gravity section, and reinforced concrete beam, presents uninviting difficulties, and Mr. Houston's attempt at this calculation for the Halligan Dam will be appreciated; but his method cannot be accepted as a satisfactory solution, and his figures do not represent the result of any logical method.

The safety of the dam is not questioned in this criticism, as it appears that the maximum concrete stress, taking the dam as an arch alone, is approximately 27 tons per sq. ft., which is moderate enough.

The author assumes for his purpose the position of the neutral axis and the stress in the steel. These two assumptions cannot be made simultaneously, as they are dependent on each other and one determines the other. Using his notation, these relations are expressed by the equation:

$$s = \frac{J - 0.5 - x}{x} \times \frac{E_s}{E_c}\, c$$

$$A_s\,(s + W) = \frac{x\,c}{2}$$

in which s is the stress in the steel,
E is the modulus of elasticity,
and A_s is the area of the steel in the section.

For the section at zero elevation, working back from the figures in the table, approximately,

$P = 81$,
$W = 77$,
$A_s = 0.00542$.

Then, assuming that $\frac{E_s}{E_c} = 15$, and $x = 10.6$, and solving, gives:

$c = 18.85$,
$s = 2.95$.

It appears that the assumption that the neutral axis is four-tenths of the distance between the compression face and the steel from the compression face as made, of itself fixes the steel stress at 2.95 tons per sq. in. instead of 6 tons, as assumed, and the tension per linear foot of dam at 2.3 tons, in place of 4.7 tons. The concrete stress becomes 18.85 tons, in place of 15.7 tons. The amount of this difference is not remarkable, but this is only because the steel present is so small in quantity as to be of little worth, and if the resisting moment is calculated, without considering the steel at all, the concrete stress of the gravity section increases only to 20 tons per sq. ft.

In a general way, the addition of a trifling quantity (here $\frac{1}{50}$ of 1%) to a masonry section, does not warrant a treatment of that section materially different from that accorded to a similar section without steel.

MAURICE C. COUCHOT, M. AM. SOC. C. E. (by letter).—This subject is one of much interest, due to the growing confidence of the Engineering Profession in the uses of reinforced concrete in dam construction. The writer was the pioneer advocate of its merit for building purposes in San Francisco, and spent much time in combating the opinions of those who were prejudiced in favor of adhering to brick construction, but its most enthusiastic advocate should take the greatest precautions to see that the cement, sand, water, reinforcing rods, and crushed rock, are of the proper standard to make a good mixture. Mr. Couchot.

Concrete construction and the uses of cement have been growing rapidly during the past ten years, the domestic output of cement having increased from 8 482 000 bbl. in 1900 to 76 549 957 bbl. in 1910.

This has necessarily created a great many new cement companies, the personnel and product of which should be carefully scrutinized by the engineer who commits his reputation to the use of their products. It cannot be alleged that any company would deliberately be guilty of willfully turning out an inferior product which would only react injuriously on the manufacturer, but it must be admitted that many cement companies—new in the business—have had to go through the mill of experience in order to produce a cement which is safe for the engineer to use. In the writer's opinion, no cement should be placed in a building or structure until it had stood the 28-day test under the supervision of the engineer, and had been subjected to some independent testing.

L. J. MENSCH, M. AM. SOC. C. E. (by letter).—Mr. Houston is to be commended for bringing up for discussion the very interesting subject of arched dams, and for having at last tried to leave the rut and take advantage of the site to adopt an arched structure. Unfortunately, the span was too large, and it is doubtful whether a great portion of the dam acts as an arched dam at all. Mr. Mensch.

Many years ago the writer investigated carefully the action of arched dams, and found that it requires the skill of a high-class mathematician to formulate the laws of the combined action of the arch and gravity principle into a workable shape. The safety of a design, however, can be checked by comparatively simple means. If a tube is subjected to an exterior pressure, the diminution of the radius due to a uniform load, p, per square inch,

$$\Delta = \frac{p\,r^2}{E\,d} \qquad (1)$$

where r is the radius, and d is the thickness of the shell, both in inches; and, inversely, $p = \frac{E\,d\,\Delta}{r^2}$.

This formula, however, cannot be applied to every section of an

Mr. Mensch. arched dam, because the deflection of the arch is greatest in the center and zero at the abutments. Although there is no deflection at the abutments, still, as in any ordinary arch, it is known that the compression there is greater than in the center.

On account of this unequal shortening of the arch, great stresses appear at the center and twice as great stresses at the abutments, and, on account of the relatively great thickness of the arch, these stresses may be even higher than those due to compression alone.

The deformation in the center of an ideal arched dam, for the various layers below the water line, may be found by Equation 1. For a structure of the dimensions of the Halligan Dam, the full line in Fig. 12 shows the ideal deformation. The same structure as a gravity

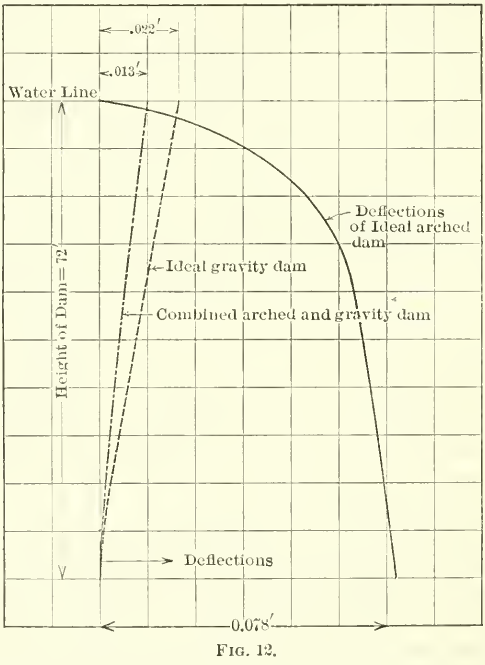


FIG. 12.

dam would deflect as shown by the dashed line in Fig. 12, if the whole water pressure were acting to deform the dam as a cantilever; and, as can be seen, this deflection is very much smaller than the deformation which would be obtained if the entire water pressure acted on the dam on the arch principle. Therefore, in a combined arch and gravity structure of the dimensions of the Halligan Dam, only a very small part of the stresses can be taken up by the arched principle.

It can be stated that the bending moment at the base, which is due to the gravity action in the combined arched and gravity dam, depends on the value of $\frac{h^4}{d^2 r^2}$, and can be obtained by the general formula, $M = \frac{p\,h^3}{n}$, in which h is the height of the dam. Mr. Mensch.

For example:

$\frac{h^4}{r^2 d^2} = 0$	0.31	0.384	0.504	1.358	9.166
$n = 6$	6.71	6.88	7.15	8.75	16.

This is only for the center section of the dam, and requires a new calculation for every other section.

There is no question that an arched structure, if it is properly designed and if the site affords proper abutments, is safer than a gravity dam alone, because water pressure entering in the joints or under the base only facilitates the arch action, even if it destroys the gravity action.

Edward L. Sayers, Assoc. M. Am. Soc. C. E. (by letter).—The striking feature of this paper is the extreme simplicity of the "analysis of stresses" which forms the basis of the design. It is not usual to base the design of so important a structure as a dam of this height on such meager calculations, but it must be assumed that the author has put this paper forward in good faith, and that the dam was actually designed with this analysis as a base. Mr. Sayers.

The investigation of the distribution of the stresses in an engineering structure is frequently the greatest difficulty confronting the designer. Mr. Houston, in his treatment of this problem, has disposed of such doubtful points by assumption. Seven assumptions are given as the basis of the analysis, and the method of the analysis itself constitutes still another, and, in fact, the main assumption. In general, all assumptions, when made the basis of a design, need defense, but this seems to have been entirely overlooked in the paper. If the design of any structure is based on an analysis involving assumptions, it should be shown, either that they approximate the truth and what the limits of error are, or, if they do not closely approximate the truth, it should be shown that the error involved is on the side of safety; that is, that the computed stresses are greater than can actually exist.

The wording of the paper ("The following analysis of the stresses in the dam * * *," etc., "Fig. 9 is a diagram of the stresses acting on the dam," "The factor of safety is about 6," and "Table 2, Summary of Stresses," etc.) would imply that the assumptions are supposed to represent at least approximate truths, and yet this can hardly be the case, because they are so evidently impossible and incon-

Mr. Sayers. sistent that they must have been thought to represent conditions on the safe side rather than approximations to the truth.

In his computations, the author assumes a fixed position of the neutral axis, the neutral axis being assumed at a distance of 0.4 of the effective thickness of the dam from the down-stream face (there is a slight inconsistency in this when taken with the assumption that the center of gravity of the compressive stress is at a point distant 0.15 of the effective thickness of the dam from the down-stream face), and the intensity of stress in the steel being assumed as constant at 12 000 lb. per sq. in. at any elevation in the dam. These assumptions can only be made together with the further inconsistent assumption that the modulus of elasticity for the concrete varies at different depths below the water surface. The following calculations show this variation of the modulus of elasticity and the unusually low values resulting. Referring to Fig. 9 (retaining the author's notation and assumptions), and further noting that his stress diagram assumes uniformly varying stress, we have the following relation:

$$\frac{12\,000}{\frac{E_s}{E_c}} : 0.6\ (j - 0.5)\ ::\ c : 0.4\ (j - 0.5)$$

or, $$E_c = \frac{c \times 0.6 \times E_s}{0.4 \times 12\,000} = E_s\ \frac{c}{8\,000}$$

in which E_c = modulus of elasticity of concrete,

and E_s = modulus of elasticity of steel, taken as 30 000 000 lb. per sq. in.

TABLE 3.

Elevation.	c, in tons per square foot, taken from Table 2.	c, in pounds per square inch.	E_c, in pounds per square inch.
40	8.0	111	417 000
30	9.7	135	505 000
20	12.3	171	641 000
10	13.5	188	703 000
0	15.7	218	818 000

The error here involved may possibly be on the side of safety; and, while it may be true that the modulus of elasticity of the concrete is not constant, may it not vary in some other manner than that selected by the author? At any rate, the assumptions made cannot result in computed stresses which are either consistent or close approximations to the probable actual, and it would seem to be unwarranted, because the assumption of a constant modulus of elasticity, instead of a constant steel stress or a constant relative position of the neutral axis, would have complicated the mathematics but slightly.

It may be noted that the author states his fifth assumption as: "That the safe load (tension) on the reinforcement is 12 000 lb. per sq. in." In that statement he does not assume that the stress in the steel of the dam is actually 12 000 lb. per sq. in., but that figure is used in all his computations of stresses, which would preclude the theory that the computed stresses are approximations to the actual, while the inconsistencies throw doubt on their safety. Mr. Sayers.

The main assumption on which the analysis is based is as follows: That part of the water pressure which cannot be carried safely by the dam considered as a beam will be carried by the dam considered as an arch. This involves a further assumption that the dam considered as a cantilever beam will have a maximum deflection at the top and a minimum deflection at the bottom, while the dam considered as an arch (referring to the intensities of thrust in Table 2) will have a minimum deflection at the top and a maximum at the bottom. For instance, in Table 2 the computed thrust in the arch is 4.4 tons per sq. ft. at Elevation 40, while at Elevation 0 it is 10.5 tons per sq. ft.; this means that the arch must deflect at Elevation 0, say, twice as much as at Elevation 40, while, on the other hand, according to the beam calculations, there is a maximum deflection at the top, decreasing to zero at Elevation 0. That these two sets of stresses, with their respective deflections, should exist simultaneously is obviously impossible. Furthermore, it is known that the dam cannot deflect at its base, and it follows (as stress must always be attended by deformation) that no such intensities of thrust from arch action as are shown in Table 2 can exist in the dam close to the base, and, therefore, the load there assumed to be carried by the arch must be carried by the beam.

It is possible that the stresses in the dam will be distributed and adjusted between arch and beam action so that the dam will be safe, but the analysis of the paper fails to show this, and, therefore, in spite of the author's statement that his design is one in which the stresses can be demonstrated to be safe, it must be concluded that his analysis gives stresses which are neither approximate nor necessarily on the side of safety.

Computations of the approximate actual stresses, based on the relative elasticity of the dam acting as an arch and as a cantilever beam, can be made, and it is believed that they should always be a part of the process of design of a dam of this character. It may be pointed out here that it is not enough to show that the dam is safe when its gravity action is completely ignored, for the fixing of the base of the dam may throw a greater load on the arch in the upper portion than is received directly from the water pressure on it.

The author's computations assume a stress of 12 000 lb. per sq. in. in the steel. When reinforcing steel is stressed to this extent, the surrounding concrete must crack. If the concrete is uniform in quality, these cracks may be distributed, and will be small, yet large enough to admit water under the full head. If, however, there exist planes of

Mr. Sayers. relative weakness, such as are likely to occur at the joinings of new work to old, or even at places where one day's work ends and another begins, these cracks will be localized and may extend to a greater depth into the dam, and even to the neutral axis, and will admit water.

The author, in Fig. 9 and in his computations, has practically assumed a crack, but neglected the force of the water pressure acting therein. The effect of this pressure would be to extend the crack; but, supposing it to extend to the author's assumed position of the neutral axis, and remembering that he assumes this position independently of the steel stress, the tension in the steel would be

$$\frac{wh\left\{0.5 + 0.6\,(j - 0.5)\right\}\dfrac{0.55\,j - 0.025}{m}}{\text{Area of steel.}} + 12\,000$$

or, at	Elevation	40	it	would	be	50 000	lb.	per	sq.	in.
"	"	30	"	"	"	75 000	"	"	"	"
"	"	20	"	"	"	46 000	"	"	"	"
"	"	10	"	"	"	61 000	"	"	"	"
"	"	0	"	"	"	74 000	"	"	"	"

These figures are not introduced as a prediction of the actual stresses in the dam, but to show to what the author's figures would logically lead, and to suggest a possible serious feature which should have been investigated.

It may be argued that, considered only as an arch, the dam may be safe, having only an average stress of about 27 tons per sq. ft., but this stress is computed under the most favorable assumption as to its distribution. It is certain that the line of thrust will not coincide with the axis of the dam, and, therefore, will not produce a uniform distribution of stress across the arch. This is especially true because the combined shortening from shrinkage, low temperature, and direct stress will have the effect of throwing the line of thrust toward the down-stream face at the abutments, and toward the up-stream face at the middle.

Mr. Houston. G. N. Houston, M. Am. Soc. C. E. (by letter).—In presenting this paper the writer had no intention of discussing the work done previous to his connection with the dam, except in a very general way. The statements regarding the cost and yardage in the foundations were based on data on record in the office of the company, which the writer assumed to be accurate. He is glad that Mr. Swigart has corrected these data from notes in his possession.

In regard to the material left on the ground by the first contractor, the writer found 800 cu. yd. of sand, 1 244 cu. yd. of quarried rock for the crusher, a considerable portion of which could not be used on account of its quality, and about 900 bbl. of cement. The company purchased all machinery, tools, and material, except cement (which was already its property) from the first contractor for $2 750, and sold the same, together with all stone, sand, and lumber on the site, reserving the caretaker's house and cement house, to the second contractor for $4 500. The cement, nearly all of which was in good condition, was sold by the company to the second contractor for $2.65 per bbl. The old quarry from which the stone for the crusher had been taken was not used, as it was in an unfavorable position, being about 50 ft. below the crusher, with a sharp uphill haul; but another quarry was opened about on a level with the crusher, with a haul which did not exceed 150 ft. It was also found necessary to open another quarry for the larger rock at a more advantageous point. Mr. Houston.

Using the profile on which the dam was finally constructed, there would have been about 12 150 cu. yd. of masonry in the proposed construction above Elevation 0. The lowest bid received for this work was $6 per cu. yd., which would amount to $72 900. In the structure as built there were 10 628 cu. yd. above the same elevation, which, at $6.25 per cu. yd., cost $66 425. Below Elevation 0 there was a saving of about 101 cu. yd. at $6, which amounted to $606, thus making a total saving of $7 081.

With regard to the analysis of stresses, the writer does not recommend the details of this discussion as the method to follow in all cases, although in this case he believes his assumptions were on the side of safety, as the results were checked by all the common methods. However, the general method of determining the safe load for the dam considered as a gravity section, and assuming that the remainder of the actual load is carried by the arch, as one extreme, and then considering the whole load to be carried by the arch action alone, as the other extreme, will indicate the limits between which the actual stress will probably lie. With the present uncertainty in regard to the actual distribution of stresses in an arch dam, no one method of analysis should be relied on to fix the profile, but great care should be taken to check the results by as many forms of analysis as possible, and also by comparison with existing structures.

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TRANSACTIONS

Paper No. 1221

PROVISION FOR UPLIFT AND ICE PRESSURE IN DESIGNING MASONRY DAMS.*

By C. L. Harrison, M. Am. Soc. C. E.

With Discussion by Messrs. G. M. Braune, Edward Godfrey, Allen Hazen, Charles E. Waddell, Rudolph Hering, M. G. Barnes, Howard J. Cole, M. H. Gerry, Jr., Edward Wegmann, Charles E. Gregory, Orrin L. Brodie, H. F. Dunham, C. Elmore Smith, J. C. Meem, W. J. Douglas, Lindsay Duncan, Arthur P. Davis, William Cain, J. W. Ledoux, L. J. Le Conte, and C. L. Harrison.

There has been much discussion recently by engineers and the technical press on the upward pressure of water, and on ice thrust in dams. The following brief statement is written to bring the subject before the Society in the hope that it will be fully discussed.

Uplift.

For convenience in discussing this subject, reference is made particularly to masonry dams on rock foundations. The principles involved will apply equally to other foundations and to dams built of other materials. The upward pressure may be due to water getting into the foundation of the dam or into the dam itself.

Foundations vary so much in character, that it is necessary to study each particular site before deciding to what extent water may get into them.

(1) In the case of a foundation of hard, sound rock, without either horizontal or vertical seams, there is no reason to expect that water will

* Presented at the meeting of December 20th, 1911.

get into it and produce an upward pressure, and, in the design, no allowance should be made for it. In such cases the junction between the masonry and the foundation can easily be made water-tight.

(2) In the case where the foundation is stratified with well-defined horizontal seams, and the dam is located near a fall or rapids in the stream, so that the water may flow from the seams at the toe of the dam as freely as it enters them from the reservoir, the upward pressure will be approximately equal to the static head at the heel and gradually decrease to zero at the toe of the dam.

(3) Take a foundation similar to the foregoing in every respect except that the water in the seams of the rock cannot escape freely near the toe of the dam, but must flow some distance down stream through rock or other materials before it reaches the surface of the ground, or must rise vertically to the surface: Then the upward pressure at the heel will be equal to the static head, and that at the toe will be equal to the head required to overcome the resistance to the water escaping at that point.

While these three cases present well-defined conditions, it is probable that at most sites the conditions will lie between those presented in Case 1 and in Cases 2 and 3, that is, the water will not be in the foundation throughout its entire area, but will cover only a part of this area. This makes it necessary to study the foundation carefully at each site in order to determine to what extent water may get into it. When this upward pressure exists, weight must be added to the dam by additional masonry to counterbalance it. Generally, it will be found cheaper to make large expenditures to provide a cut-off in the foundation, which will not only reduce the uplift, but will also save the water. Such a cut-off should be located at the heel of the dam. If it is located under the middle of the dam, there would be an upward pressure under the up-stream half of the dam, due to the full head of the water in the reservoir.

A thorough investigation, by borings and otherwise, should be made of the foundation at each site before the dam is designed, and a liberal margin should be allowed over what the engineer (basing his figures on his experience and best judgment) believes to be safe.

In order to determine what allowance to make for pressures due to water which gets into the dam itself, one must first decide on the character of the construction. With suitable stone, sand, and cement, it is possible to build a masonry dam which will have no horizontal cracks or seams, and it is also possible to provide against vertical cracks, to a large extent, by expansion joints. Water in vertical cracks, however, does not produce an upward pressure. In such structures very little, if any, allowance should be made for the upward pressure due to water getting into the masonry.

If the materials for building water-tight masonry are not to be had at the site of the dam, and it is very expensive to import them, it is generally advisable to adopt a different class of masonry, which will probably be more pervious and also more difficult to construct without horizontal cracks or seams, thus allowing the water to enter the dam, and resulting in upward pressures. The extent of such pressures will depend on the character of the masonry and the care with which it is built, all of which must be known before an estimate can be made of the extent to which the water will get into the dam. The effect of this upward pressure, however, must be counteracted, either by increasing the section of the dam or by increasing its height above the water level in the reservoir, or by both. In many cases it may be advisable to provide drainage wells near the up-stream face to intercept the water and carry it off through pipes at the toe of the dam, thus reducing or eliminating its effect in the main body of the dam. After determining the type of masonry to be constructed, it is still a question of judgment, based on observation, tests, and experience, as to what the upward pressure in the dam will be.

The upward pressures in the foundation, and in the dam itself, should be considered separately before a decision is reached.

Ice Pressure.

After ice has formed on a reservoir, it contracts under a lower temperature and expands under a higher temperature. The contraction due to the cold weather of winter results in cracks which fill with water that freezes and produces a continuous sheet of ice over the surface of the water. Under the higher temperatures of the late winter and early spring, the ice is warmed up and expands. On reservoirs this results in the ice being forced up the banks, unless the inflow of water should raise the level sufficiently to provide the increased area. In case the banks are vertical or nearly so, the expansion produces a pressure on the sides of the reservoir and results in the ice being compressed to some extent and buckling up out in the reservoir. In some cases the ice may be crushed.

In designing dams, the ice pressure should be considered, but it is of less importance than the upward pressure of water in the dam and its foundations. The ice is in sight and can be cut along the face of the dam, and thus relieve the pressure. Generally, in storage reservoirs, the period of heavy ice pressures is also that of low water, and the pressure would come against the dam at a point considerably below the high-water level, where the dam is strong enough to resist it. In many cases, the dam is located in a narrow gorge where the full effect of the ice field cannot reach it. The dam should be strong enough to resist the water pressure and the additional pressure caused by the ice, but what this will be depends on the thickness of the ice and other local conditions at each site, and no general rule can be made to cover all cases.

DISCUSSION

Mr. Braune. G. M. Braune, Assoc. M. Am. Soc. C. E. (by letter).—This paper is a timely presentation of the subject of uplift pressure on masonry dams.

Assuming a solid rock foundation and a certain head of water, different designers will arrive at quite different sections, owing to the varying assumptions in their calculations. The width of the bottom of a solid masonry dam will depend on the position and direction of the resultant force, and if the intersection of this force with the base is kept well within the inner third, and the angle of inclination with the normal is small, the uplift force will be cared for automatically in the calculations. As it is probable, however, that the uplift force does exist, it is proper to provide for it in the design. Whether the full static head is exerted over the entire base or diminishes in magnitude toward the down-stream edge would depend on several conditions, namely, the nature of the rock formation, the character of the workmanship, the method of laying masonry, etc.

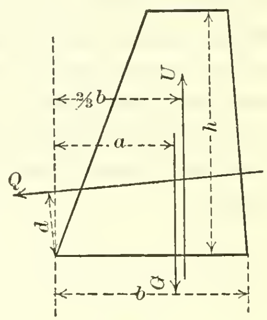


Fig. 1.

In Fig. 1, the assumption is made that the full static head is exerted at the up-stream edge and is zero at the toe of the dam. This uplift force is then, $U = \frac{h\,w\,b}{2}$, in which h = the head of water, w = the weight of 1 cu. ft. of water, and b = the width of the dam at the base. The resisting moment is $G\,a$, and the overturning moment from the lateral water force is $Q\,d$.

If an allowance of F is made for a factor of safety for the overturning moment of the lateral force, and the uplift is taken into account, then the equation for stability against overturning will be:

$$G\,a = F\,Q\,d + \frac{b^2\,h\,w}{3}.$$

In Fig. 2, in which the full static head is assumed, the equation against overturning will be:

$$G\,a = F\,Q\,d + \frac{b^2\,h\,w}{2}.$$

Whether the first or second condition is assumed in the calculations, how then should F be chosen?

In the design of structures of steel, reinforced concrete, and similar materials, the agreement among the authorities as to the size of the factor of safety is fairly uniform. Of course, in designing dams, this factor cannot be fixed so definitely, but, for the same conditions, there should be some uniformity. Mr. Braune.

If this factor is assumed as 2, the base of the dam will be somewhere between 70 and 80% of the height in the first case; and in the second case, with full static head, 80 to 90% of the height. In the design of the dam at Marklissa, across the River Queis, in Prussia, the designer, Intze, assumed the full static head as acting over the entire base of the structure. The dimensions are: Width of crest 18.7 ft., base 123.6 ft., height 141.0 ft. It is founded on impervious rock.

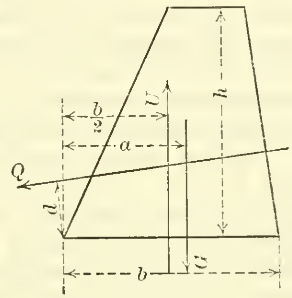


Fig. 2.

Then, again, what should be the maximum deviation of the resultant from the normal? To provide for a small deviation will require a large base, if the full static uplift is assumed. Some designers require that the masonry should be built in alternate blocks of from 50 to 60 ft., in order to provide for expansion and contraction, while others ignore this requirement, and vertical cracks result. If these cracks are not harmless, why should the owners be subjected to the additional expense of alternate block construction?

It is to be hoped that, before discussion on this paper is terminated, many controvertible points in the design of dams will be thoroughly debated.

Edward Godfrey, M. Am. Soc. C. E. (by letter).—One of the most momentous questions at present before the Engineering Profession concerns the design of dams; not that it inherently possesses such serious problems, but that the question is a psychological one. When a large body of men is compelled to change any adopted ideas or standards, something must happen. Even in so simple and eminently useful a change as the adoption of standard time in place of sun time, bitterness and strife amounting almost to revolution accompanied the discarding of "God's time." Many towns for years had two or three standards of time. Thus far it has not been held that the (to date) general standard of designing dams is of divine origin, but something of the same fanatic opposition to discarding it has been heard from many quarters, in spite of the many lives which have been sacrificed to it and by it, as was heard when "railroad time" was on the rack; and compromises are suggested, as in the other case. Mr. Godfrey.

It is absolutely certain that engineers must revise their methods of calculating the stability of dams; the serious question is whether or not that revision will be thorough and complete, or will it for a

Mr. Godfrey. season of years be a dangerous compromise to be completed only when another great disaster and the blood of other scores or perhaps hundreds of victims cry out? No future book on dams will commit the awful blunder of all English books written previous to the first failure of the dam at Austin, Pa.; no future book will omit entirely all mention of upward pressure on masonry dams. Surely no revisions can be expected to lay adequate emphasis on this omission; and it is doubtful if many new books will be written on the subject, as it is one which engages the active attention of only a few men. Furthermore, in some quarters, there is a tendency to belittle the real and demonstrable importance of the matter. The writer considers Mr. Harrison's paper an example of this tendency.

Railroad bridges could be built with no regard whatever for the dead load in calculating the strains. The majority of them would stand up and do service. Some of them would last a long time and never give the least sign of their inadequacy. Some of them, in fact, might never be over-stressed, as the assumed live load is usually in excess of any train load. There would be an occasional wreck, but as the fracture of steel is always sharp and crystalline in a sudden break, any required number of "experts" could be found to prove that the steel was burnt in the manufacture or crystallized in constant service. Just as experts of this class can always find a bad batch of concrete, a shaving, a block of wood, or some dirt in any reinforced concrete failure, so they can detect defective steel or workmanship anywhere.

If every book on the design of bridges omitted all mention of the need of considering the dead load, and gave examples of designing which ignored the dead load, and if some "destructive critic" should come along and question the correctness of this, even maintaining that it was positively wrong and wreck-breeding, there would be the same hue and cry against this "iconoclast" that the under-pressure advocates have met; and he would be treated with the same silent contempt from some parties until those same parties had had time to get under cover gracefully.

It is as erroneous a proceeding to design masonry dams of any class without considering the under-pressure as to design bridges without considering the dead load; but it seems to be more disquieting to many to have accepted standards which have proven false than to have the Engineering Profession degraded by the periodic wrecks that these same false standards engender.

The foregoing is preliminary to this thesis: All masonry dams should be designed capable of withstanding upward pressure under the full area of the base, the intensity at the up-stream edge being not less than the full head. The writer will go a step farther and say that legislation ought to be passed requiring all masonry dams to be thus built, just as it requires buildings to be designed for dead and live load. Mr. Godfrey.

The wrecks which are occurring with such sickening regularity are writing against the Engineering Profession: *"Mene, Mene, Tekel, Upharsin."* It is time for the Medes and Persians to come in with some laws which cannot be altered. These three laws would have avoided nearly all the great structural wrecks known:

(1) Dams must be built to resist upward pressure.

(2) Structures must be substantially braced during and after erection.

(3) Concrete shafts must not be considered reinforced when they contain only slender vertical rods, even if these rods are wired together at wide intervals.

To be asked for a demonstration of the proposition that water exerts an upward pressure under the base of a dam and in horizontal joints is like being asked to demonstrate that the earth is not flat. To the writer's mind it ought to be all-sufficient if mere mention is made of under-pressure as a factor working against the stability of a dam. No demonstration is ever attempted to show that full water pressure against the up-stream face of a dam must be considered, though that face may be largely covered with mud and silt. One might make a paper demonstration which would show that a fillet of mud on the up-stream side of a dam would save a large quantity of masonry, and he would stand on precisely the same ground as the engineer who maintains that under-pressure may be neglected.

The failure of the Austin Dam, which occurred in September, 1911, has brought the subject forcibly before the Profession. Here was a dam of standard proportions, which, by all the books on dams, was capable of withstanding the full pressure of the impounded water. It failed by reason of the pressure of water under it. This is just as clear and undeniable as the fact that the thing which causes the great force in a hydraulic jack is the small pump which exerts a pressure on a very small area of water.

Proof of the fact that water will exert pressure wherever it is confined and in communication with other water reaching to a higher head is too puerile to demand attention. Proof of the fact that there is water in the joints of a dam and beneath it, and that this water meets the last named condition, is equally puerile. Some one may say, in answer to this, "capillarity!" Is it capillarity which taxes the capacity of several steam pumps to keep unwatered the foundation of a dam during construction? Is it capillarity which causes water to well up in a spring apparently out of the solid rock? Is it capillarity which carries water a mile or more through compact earth and causes

Mr. Godfrey.

it to rise in wells or basements to just the height of the surface of a river in the neighborhood? Is it capillarity which causes water to flow out in jets through the joints of a dam, or to seep through the soil beneath a dam and force its way to the surface? Is that water discriminating enough to avoid the base of a dam in its passage? Is it capillarity which causes water to ooze out through a cast-steel cylinder (a little spongy) under great hydraulic pressure, when the same cylinder would be quite water-tight under ordinary high pressure? Is it capillarity which will eventually force out a tightly driven plug in the orifice of a house faucet, if the faucet should leak a few drops a minute? Was it capillarity which forced water under very low head through 30 ft. of solid concrete, as shown in the report of the Chief of Engineers, U. S. A., for 1902?* Is it capillarity which lifts the water-proof skin applied to a damp wall?

Mr. Harrison would have us believe that one of the easiest things in the world is to exclude water from a masonry wall, to place masonry on solid rock in such a way that no water will ever enter, and finally to find the solid rock without seam and of indefinite horizontal and vertical extent and to know of its existence before the excavation is made or the plans decided on.

In a recent book† occurs the following:

"Mr. J. B. Francis held that solid concrete deposited on bed rock would be lifted or floated, and to prove this, placed a pipe provided with a gauge, in the concrete of a dam and found that the gauge registered the full pressure."

The writer does not find a record of this experiment by Mr. Francis, but a test that amounts to the same thing is recorded,‡ in which practically the full pressure of water was communicated through 18 in. of carefully made cement mortar. When the pipe was left open, this same cement mortar allowed only about a bucketful of water to pass through in a day under a pressure of about 70 lb.

When engineers want to exclude water from a foundation, they cover the wall or ground with several layers of tarred felt. When they want to make a masonry cistern to hold water, they carefully line the entire inside surface with the same material. Do they ever line the up-stream face of a dam in the same way? Do they ever paper the bottom of a stream or lake?

Concrete can be made water-proof by making it wet and pouring it continuously; but the mortar used in laying ashlar or rubble masonry is not water-proof, because it must be a drier mixture in

* In *Engineering News*, April 2d, 1903, p. 306, Col. Peter C. Hains states: "This showed conclusively that there was less resistance to the passage of water through the 30 ft. of concrete than to its passage through the sandy material forming the earthen portion of the parapet."

† Beardsley's "Hydro-Electric Plants," p. 247.

‡ *Transactions*, Am. Soc. C. E., Vol. XIX, p. 147.

order to be handled properly and to set in a shorter time. This means that it has something of the properties of the old-style concrete, which acts more like a filter. Furthermore, the under side of a stone laid ever so carefully on a prepared bed of mortar will in all probability have many cavities. In some experiments by the writer in placing cast-steel column bases, grouting through the openings therein left large air spaces, as noted when the castings were removed for examination. It was only when the cement mortar was carefully mounded and the bases brought down with their first contact at the crest of this mound that these air spaces could be eliminated. Mr. Godfrey.

Efforts to prevent water from penetrating masonry walls are strenuous enough, in all conscience, to convince engineers that the only safe course is to assume that the water will penetrate a dam, and to provide for its maximum effect. Horizontal joints between two successive days' work in concrete are planes of entry for water. There are many evidences of this in copious leaks in walls and dams. The entire section of a concrete dam, therefore, should be designed accordingly, for such joints may or will always exist.

In a recent issue of a technical journal* there is a picture and a description of a concrete reservoir 100 ft. in diameter and 40 ft. high. Water is pouring out or has poured out of practically all the horizontal joints. The thickness of the shell is as much as 42 in. This reservoir was subsequently water-proofed by the expenditure of 900 bbl. of cement, 15 bbl. of water-proofing compound, 20 tons of steel, 500 cu. yd. of broken stone, and $11 000; and yet some builders of concrete dams will fondly imagine that no water ever penetrates the concrete or ever exerts any pressure beyond the line defining the up-stream surface.

It is true that seepage of water through concrete will act to seal up the pores through which it passes. This is not a filtering action, necessarily, but may be purely chemical. It is possible, and in fact probable, that the action begins at the surface which is farthest from the water. This would supply exactly the condition that would intensify the pressure. It is entirely possible that the water would evaporate on the surface of the concrete as fast as it seeps through, so that no leak would be apparent. A porous surface is an admirable one to promote evaporation. In southern countries this principle is utilized to keep water cool. Porous earthenware jars containing water are hung up in the breeze so that the evaporation of the water which works through will cool the contents of the jar.

Dams have been classified in the matter of height and alleged cause of failure, with a view of showing that under-pressure is not responsible; but one thing is strangely absent in these classifications, namely, information showing that any of them would be stable against

* *Engineering Record*, August 19th, 1911.

Mr. Godfrey.

this under-pressure. A boulder will lie on the bottom of a river and remain stationary against its current. The fact cannot be gainsaid that a solid block of concrete laid on the ground may be made to hold back water on one side of it, though it may rest in ordinary mud. Failures of foundations are exhibited by sinking; failures of dams are usually shown by the sliding of large blocks after they have been lifted and lubricated by water under them. The character of the foundation has little to do with the existence of the under-pressure or the need of providing against it; though if failure takes place, due to under-pressure, it will be more complete and disastrous if the foundation is soft and yielding than if it is a solid rock. It may be even true that one dam designed without regard to under-pressure and built on a solid rock will stand up, whereas another built on a poor foundation, also designed without regard to under-pressure, will fail. This cannot be construed as an argument for disregarding a fact. Analogous to this, it is quite possible that one bridge designed regardless of the dead load will stand, whereas another, of longer span, will fail. In the case of bridges, the dead load in the long span is of greater importance because of larger amount, both relatively and absolutely. In the case of dams, the under-pressure on the dam resting on yielding soil is of greater moment because concentration of the pressure on a yielding base is more apt to cause failure, also because the softer soil will allow freer sliding. In neither case can the lesser menace be disregarded with impunity or with reason.

One of the arguments which is supposed to show that, of the 30 or 40 masonry dams which have failed in the last 20 years, under-pressure has not been the cause, is the fact that blocks generally slide out and do not overturn. When under-pressure, assisted by the horizontal pressure, has pried a dam loose, the former has spent the greater part of its force. It would require time in order to gain a new momentum, as the water can enter but slowly in a narrow slit. The escape of a very small quantity of water in a test under great hydraulic pressure drops the gauge pressure very quickly. In the case of the dam, there is the ever-present horizontal pressure, with practically unlimited volume behind it, and this quickly acts to force the dam out in a horizontal direction.

Cut-off walls are a "delusion and a snare," except for the legitimate use of reducing loss of water. They do not inhibit under-pressure, for water may pass through a long and circuitous course and, at the end of the same, if confined, exert its full pressure. A certain dam* had two cut-off walls, one 8 ft. deep at the up-stream edge and the other 5 ft. deep at the down-stream edge; it was completely under-

**Engineering News*, April 1st, 1909.

washed. Another dam* had two cut-off walls, the deeper one being at the down-stream edge; it failed by under-pressure. Mr. Godfrey.

Under-drainage is another broken reed. To be effective, the whole dam would have to be honeycombed, and much water would be wasted. It is also likely to silt up or freeze and be rendered useless. Furthermore, it would be expensive. The same amount of money spent for additional concrete would be of more lasting good.

Anchoring into the rock to make up for deficiency in stability is another feature which ought to be condemned most severely. It is very difficult to anchor an ordinary rod in rock for its full tensile strength, and it is impossible to ascertain whether or not it is thus anchored.

For seven years the writer has been endeavoring to bring the subject of the design of dams before the Engineering Profession in an effort to break the solid front presented against safe design. He earnestly hopes that the agitation which the Austin failure has evoked will result in the complete rejection of the insane and unsafe method —the general standard which ignores under-pressure—and the adoption of the only safe and sane course, namely, to design all masonry dams for under-pressure, irrespective of the opinion of the designer. A designer's opinion never yet has sustained a weak structure. The more correct rules are adopted, and the less need for individual opinion, the safer will be the results.

Allen Hazen, M. Am. Soc. C. E. (by letter).—The existence of upward pressure under a dam depends on the relative perviousness of the material under it at different places, rather than on the absolute quantity of percolation. The cut-off works under the heel of the dam may be nearly tight, but they will never be entirely so, and if there happens to be some place down stream where the material is entirely tight, seepage through the cut-off, however slight, will result in the production of a pressure under the base of the dam, equal to the full static pressure of the water in the reservoir. Of course, this extreme case is not to be expected, but the material under the toe may be less pervious than that under the heel, and, in that event, the pressure under the dam may be a considerable proportion of the whole static pressure of the water. Mr. Hazen.

If the material under the dam is obviously open and pervious, or has seams, the construction of a cut-off under the heel seems desirable and necessary, and, of course, it should be made as nearly water-tight as possible; but how is the engineer to know that there is not another place down stream, where the material may be sufficiently impervious to hold back whatever seepage there may be through the cut-off, and thereby produce upward pressure under the dam?

* *Engineering News*, January 13th, 1910.

Mr. Hazen. The same line of thought, of course, applies to any point in the body of the dam with respect to the masonry in that part of it below the point. Even though special care is taken to make the masonry near the upper face as impervious as possible, it may happen that, sometimes, some other place will be less pervious. Perviousness is not sufficiently under the engineer's control to prevent this from happening. In so far as it does happen, internal pressure will result, which is, in effect, upward pressure on that part of the dam above. Making the upper face of the dam as nearly water-tight as possible, and building cut-off works under it, may generally render this part of the work less pervious than the rest, but it cannot be safely depended on to do so in all cases.

A further line of defense is desirable, and is usually possible. This consists of drainage channels to facilitate the flow of water from the down-stream part of the dam, and from the rock under it. If such drainage can be made sufficiently comprehensive and certain in its action, it eliminates the possibility of upward pressure at any point controlled by it.

It is easy to build drains vastly greater in carrying capacity or perviousness than the part of the dam near the upper face and the material under it. Such drainage has been carried out in many dams. In the Cataract Dam, for the water supply of Sydney, Australia, the upper face of the masonry, to a depth of 2 or 3 ft., was built with especial care, and this alone was relied on to hold the water. The remainder of the dam was built of good, though more pervious masonry, and throughout the whole of it were placed 6-in. rectangular conduits, filled with broken stone, parallel to and about 6 ft. back from the up-stream face. These are collected into 6-in. earthenware pipes, laid at right angles to the longitudinal axis of the dam, with exits on the down-stream face.

In the Ashokan Dam, of the New York water supply, open passages, of sufficient size for men to pass through and observe the conditions existing in various parts of the structure, have been left.

By eliminating the possibility of upward pressure, this system of drainage adds greatly to the stability of the structures protected by it, and it would seem well to apply it to all large masonry dams, depending mainly on their weight for stability. In small dams, it is relatively more difficult to apply it, and the protection is less complete, because there is a certain minimum distance from the upper face within which drains cannot well be extended, and this minimum is a larger percentage of the whole thickness of the dam when the structure is not large.

In short dams, also, it may be easier to brace against the rock sides of the valley as arches, and make them strong enough in this way, rather than carry out special precautions against water pressure below. Mr. Hazen.

It is especially interesting to note that the late James B. Francis, Past-President, Am. Soc. C. E., in a paper* presented May 16th, 1888, discussed the need and advantages of drains to prevent the occurrence of water pressure in and under dams, and advocated the use of such drains. After the lapse of more than twenty years, little can be added to what he then said, from a theoretical standpoint; but, in that time, there has been experience with a number of dams, which adds force to the statements then made.

Charles E. Waddell, M. Am. Soc. C. E. (by letter).—This timely paper is most suggestive. Those who have followed the discussion of masonry dams appearing in recent numbers of the technical papers must have been impressed with two things: First, the wide differences of opinion as to whether or not under-pressure is to be considered; and second, that nearly all writers confine their discussion to dams of the reservoir type. As to the question of under-pressure, opinion appears to grade down as evenly as the pressure itself, from the conservatism of John R. Freeman, M. Am. Soc. C. E., who would consider the entire pressure acting uniformly under the whole dam, to the extreme of Edward Wegmann, M. Am. Soc. C. E., who disregards under-pressure where the foundation is at all trustworthy, and cites as examples a number of dams now standing. One writer states that five or six high dams have failed in the last ten or twenty years. From very meager statistics the writer finds the record of some thirty overflow dams in the same period. The average height of these dams was, perhaps, from 30 to 40 ft. Just why the greater number is of this type and size is purely conjectural. It may be that the failures are proportionate to the total number of the two types which have been built; or it may be that fewer high dams fail, owing to superior engineering and construction in works of greater magnitude, or that the forces tending to destroy the overflow dam are greater and more numerous than those acting on the high reservoir dam, and are less appreciated and understood. It may be pointed out that a dam with a surcharge has to contend with at least three factors peculiar to itself: First, the possibility of a vacuum caused by the sheet of falling water; second, the shock and vibration of the water in motion; and third, the effect of the body of water at the toe, which, assuming penetrability of the masonry, may tend to cause flotation. It is generally conceded that the first and second factors can be counteracted to some extent by giving the down-stream face of the dam the shape of the under side of the sheet of water at the maximum expected surcharge. Mr. Waddell.

* "High Walls or Dams to Resist the Pressure of Water," *Transactions*, Am. Soc. C. E., Vol. XIX, p. 147.

Mr. Waddell. In theory this would be a parabola; but in practice the end sought is obtained by giving the masonry easy curves of varying radii approximating the calculated parabola. If there are any uneven internal stresses in the masonry caused by temperature changes, retarded rate of cement setting, etc., the jar and shock of the falling water would seemingly be a most plausible explanation of some failures. In some dams of the overflow type recently built, the weight or holding-down effect of the falling sheet is stated to have been considered, together with the back pressure of the water below the dam. This last appears reasonable; but to pin any faith on the weight of the falling sheet would seem illogical, if the face of the dam is virtually the path of the particles of water in motion.

The wide variance of opinion as to what factors bear on the design of dams; the undeniable qualification that such a structure should be safe; and the further equally necessary stipulation that the cost be not prohibitive; all suggest the desirability of the Engineering Profession coming to some general agreement as to what constitutes safety, and deciding, as far as generalization is possible, the factors and limits governing conservative design.

While it is probably a mere coincidence, and is not even suggested as anything more, in a number of hydro-electric plants where the dam is approximately 35 ft. high, the writer has observed that the cost of the dam averaged about $20 per developed horse-power, as compared with perhaps $10 for water-wheels and $10 for electric machinery. Following Mr. Freeman's policy of designing for full hydrostatic pressure under the dam, it is not at all improbable that the preliminary estimates would have indicated the project as too costly to be remunerative, whereas the dams are standing and the plants are earning a fair return.

Two features of the present controversy are regrettable: the desire in some quarters to blame the corporations for insecure structures, and the clamor for political supervision of dam construction. There may be some companies which are avaricious to the point of taking a gambler's chance in order to save a few dollars, but, in all the dealings the writer has ever had with a corporation, he has yet to meet the case where greed has mastered sound reasoning; and if an engineer is so unfortunate as to be placed in such a predicament he can resign; nobody can force him to do wrong; hence the writer cannot see where the blame attaches to the corporations.

The writer suggests the wisdom of proceeding with great caution in endorsing, as a Society, or as engineers, the appointment of any State boards or commissions to pass on dam construction. Our national panacea for any and all evils is legislation; but since engineers, as a body, are not agreed on the salient features of dam design, it is expecting too much, is it not, to hope that a commission could strike the happy mean by which the public would be secure and the owners of a dam not suffer from excessive cost? Mr. Waddell.

In closing, it may be pointed out that while many engineers, including the writer, are prejudiced, perhaps unreasonably, against the reinforced concrete dam, certainly the discussion of the present time strengthens the position of the advocates of this type of construction.

Rudolph Hering, M. Am. Soc. C. E.—Mr. Harrison's suggestions are so complete, and Mr. Wegmann has explained the subject so fully, that it is difficult to add much to the discussion. Mr. Hering.

It has always seemed to the speaker that the two questions depend entirely on local conditions. There are cases where neither uplift nor ice pressure need consideration, and there is no doubt that under some circumstances they are exceedingly important.

The modern tendency is to look out for the upward water pressure to a greater extent than formerly, because of the exercise of this pressure through open seams. The remedy in the case of a dam is proper drainage, and this can generally be provided in a simple and effective way. The upper side of the dam should be made as water-tight as practicable, in order to prevent water from entering, but if any should enter the main body, it should be given an opportunity to drain away and not produce the otherwise resulting upward pressure. This drainage should be provided for the body of the dam as well as for its foundation, should this be necessary.

There has been more experience, perhaps, in the effects of this upward pressure in reservoirs than in dams. The bottom of a small reservoir built on one of the Hawaiian Islands was lifted and broken by such pressure. This result was due to percolation through coral rock, which, according to all appearances, should not allow much water to pass through except in very small channels. A very fine stream of percolating water, however, if it is free to move, is sufficient to exercise this upward pressure. It is the same old phenomenon which by the Greeks was called the hydraulic paradox, when it was realized that by a very small quantity of water a very large pressure could be produced.

M. G. Barnes, M. Am. Soc. C. E.—The discussions on Mr. Harrison's paper have been confined to the foundations and the upward pressures under the dam at the foundation; but, some points which were not mentioned appeal to the speaker as having great weight in the design and construction of concrete dams. Mr. Barnes.

In depositing large masses of concrete, horizontal joints are frequently permitted; in fact, even if prohibited by the specification, it is not always possible to avoid them, because storms, failure of plant, Mr. Barnes. failure of delivery of materials, etc., may make such a joint necessary, and at a very undesirable plane. Moreover, in the methods frequently practiced, very wet concrete is deposited, and large quantities of water are permitted to accumulate on top of it. This water carries the inert matter and laitance from the sand and cement, which is deposited on top of the concrete. The speaker has seen such a deposit fully 1 in. thick, with new work built directly on it. This deposit prevents any bond between old and new work, and allows hydrostatic pressure to take effect at the joint. It may be argued that such work should not be permitted, and that the laitance should be removed. It is well enough to lay down drastic rules in the specification, but the designer or consulting engineer is not usually the constructing engineer, and his orders are not always carried out. Therefore, such poor construction must be taken into consideration in the design.

If, under these conditions, the full upward pressure, as has been suggested, and the high ice thrust are assumed, in addition to the ordinary horizontal pressure, absurd results will follow. For example, take the condition mentioned by Mr. Brodie, but consider a spillway section with water and ice just at the coping level. Assume a full upward pressure on the joint which is 10 ft. below the coping, and a horizontal ice pressure of 21 500 lb. The speaker has computed the stability of the dam under these conditions, and, unless he has made some mistake, the weight of the masonry above the 10-ft. joint is approximately 18 500 lb. This is less than the assumed horizontal pressure of the ice alone. Assuming a full upward pressure, the effective weight of the concrete is only about 8 200 lb., and its resistance to sliding, using a coefficient of 65%, is 5 330 lb.; but the sum of the assumed horizontal pressures is 24 625 lb., or about 4.6 times the resistance of the dam. In other words, the dam would have to be about 50 ft. wide at the spillway in order to balance the sliding forces. Manifestly, the conditions assumed are not realized in practice. It is just as reasonable to make these assumptions for the horizontal joints as for the base of the dam; in fact, the bond between the rock and the concrete is better than at the joints.

Mr. Gregory has assumed that the water stood at the elevation of the coping, and has discussed a joint 10 ft. below. If it is assumed that the dam is built 10 ft. above the ice pressure, then the only pressure exerted on the dam at 10 ft. below the coping, is that from the ice, and the total force tending to slide the dam is 21 500 lb., while the resistance to sliding, assuming a coefficient of 65%, is 12 000 lb.

Mr. Cole. HOWARD J. COLE, M. AM. SOC. C. E. (by letter).—In the design of a masonry dam, the necessity for making provision for ice pressure is more theoretical than practical, as witness the many dams now in use for which such provision was not made; the designer doubtless being influenced by the fact that as the body of water impounded by the dam would have sloping shores which would care for the expansion of the ice, the vertical face of the dam would form a very small proportion of the shore line, and provision for ice thrust need not be considered. Mr. Cole.

In one large dam, the impounded lake is about 30 miles long, making an approximate shore line of 60 miles, of which the dam itself forms some 1 200 ft., or about $\frac{1}{263}$. This same condition is true in the case of most large masonry dams, though not in this ratio, and it is seen that, except where the shores are vertical, Nature has amply provided for the expansion of the ice.

An important feature which has a decided bearing on the necessity for a provision against uplift, and is frequently seriously neglected, is the need of a careful exploration of the subsurface condition of the dam site before designing the structure.

The mere fact that rock crops out along the dam site is no assurance that the location is a proper one on which to build. Core borings should be taken at frequent intervals over the site extending well down into the bed-rock (the depth varying with every location, and depending on the geology of the country), in order to develop fully the character and stratification of the underlying rock.

In a limestone country, the rock is likely to be permeated by cavities and subterranean passageways which would be disastrous to a dam built over them, and would surely cause some uplift, for which provision should be made by completely filling the voids with grout and concrete, and adding to the dam section.

A proposition to build a dam in a region similar to that above described was reported against for the reason that the passageways (discovered by careful investigation) proved to be so numerous and of such large section that it was not feasible to construct a safe dam for the proposed expenditure.

There is now under construction an important dam in a limestone section where considerable trouble has been caused by these same conditions, and a large expenditure is being made to discover the subterranean cavities in order to fill them with grout.

Sandstone, because of its nature, is likely to be very porous, and, with its seams and strata, affords a poor foundation unless provision is made to eliminate these dangers. A careful system of borings at the site of the dam at Austin, Pa., would have disclosed the porosity of the rock, and would have prevented the disaster, as suitable provision would have been made in the design to counteract the conditions imposed by poor foundation rock. Too much emphasis cannot be placed on the necessity for careful preliminary subsurface exploration.

Mr. Gerry. M. H. GERRY, JR., M. AM. SOC. C. E. (by letter).—Mr. Harrison has clearly stated certain elements to be considered in connection with the design of masonry dams. "Uplift," as commonly used, refers to the vertical force acting upward under a dam or on some plane of cleavage in the structure itself. This force results from hydrostatic pressure either from the head-water above or the tail-water below the dam. There have been failures of dams caused by high back-water at times of flood, and as this condition is common to many structures, it should be considered. The author has referred to "uplift" as produced only by pressure from the head-water, but where there is considerable submergence of the masonry by back-water, a substantial force results therefrom. The uplift resulting from the pressure above can be reduced and limited very materially by the proper design of the structure itself. Every important masonry dam should be provided with a thorough system of drainage; and this should include the masonry, the foundation contact, and the bed-rock to a considerable depth. All drainage water, and especially that from the bed-rock, should be brought into a chamber or tunnel extending through the dam where the flow can be observed and its quantity determined. A complete drainage system adds but little to the cost of an important structure, and, if properly designed, it will prevent heavy uplift, which otherwise may result from pressure above the dam. The uplift from water below the dam cannot be limited in this way, and should always be considered with the other forces.

Even in the author's Case (1), where the foundations are hard and sound, drainage should be provided, in order to limit the upward pressures. There is no such a thing as a water-tight foundation in the sense that sufficient water will not pass to produce pressure, and the only way to prevent this is to give the leakage a free discharge. In the author's Case (2), where the bed-rock has horizontal seams and there is a fall or rapids below, it is assumed that the uplift will be equal to the static head at the up-stream face, and zero at the heel of the dam. This is hardly a safe assumption. It is better to drain the structure and the bed-rock artificially, first having prevented, as far as possible, all leakage through the rock by a deep and well-constructed cut-off wall, and by grouting with cement the bed-rock itself. The small remaining leaks can then be brought out through suitable drainage pipes and but little uplift will result from the head-water pressure. The same remarks apply to the author's Case (3), save that it is well to observe that the full static pressures should always be assumed for a limited area of the base adjoining the up-stream face of the dam, and the pressure due to the maximum head of water below the dam should be assumed as acting on the remaining area of the base.

The preparation of the bed-rock is of the very greatest importance, and the judgment of the engineer must here be conclusive. As a rule, important dams should be deep seated into the rock and not merely placed on the upper surface. As far as possible, the masonry should be made one with the foundation rock, so as to avoid large and well-defined areas of cleavage. To accomplish this, it may be necessary to excavate the bed-rock to different depths. The author has well said that thorough investigations of the bed-rock at the site should be made by borings and otherwise. If the rock is at all soft and seamy, it should be investigated by borings to great depth, and should be tested for tightness by applying water and air pressure to the drill holes. If the dam is to carry considerable head and the rock is found not to be thoroughly tight, then it should be sealed with cement grout forced in under pressure; this operation being repeated until test holes show that it is substantially tight, after which drainage holes should be drilled into the rock for a moderate depth, and these holes should be piped and connected with the drainage tunnel. In the case of seamy rock foundations, a deep cut-off wall is of the utmost importance, and there is no reliable substitute therefor. If the rock is soft, the excavation for the cut-off can best be made with a channeling machine so as to avoid disturbing the adjoining rock. The grouting of the bed-rock, previously referred to, can be done with advantage after the cut-off wall is in place. Mr. Gerry.

Every large dam should be provided with a reasonable number of vertical joints. The writer prefers to call these adjustment joints rather than expansion joints, because their principal function is to take care of the adjustment which takes place in the dam as a result of the setting of the concrete and the establishment of final temperature conditions within the body of the masonry. In a massive structure, such as a great dam, it requires a very long time (it may be years) before permanent conditions are established. Changes of atmospheric temperature have but little effect on a massive dam, except near the surface. In some cases, in very cold climates, and especially if there is no water flowing over the dam, it is well to reinforce with steel near the surface in order to prevent local cracking, which, followed by the entry of water and repeated freezing, may otherwise injure the surface. The adjustment joints, however, serve an entirely different purpose, as they take care of the shrinkage of the concrete and prevent cracks running in irregular directions through the masonry, which otherwise are certain to occur. In cold climates, ice pressure should always be taken into account. In an important structure it is not safe to assume that the ice can be kept clear of the dam by maintaining an open trench. The right thing to do is to assume the full pressure equal to the crushing strength of the ice

Mr. Gerry. applied at the points where it will rest after the dam is in service. In designing a dam, all the forces acting should be taken into consideration, as in any other engineering structure. If this is done, and proper attention be given to the workmanship, the materials of construction, the preparation of the foundation, and the various details of design, there is no reason why structures of this kind should not be among the most enduring built by man.

Mr. Wegmann. EDWARD WEGMANN, M. AM. SOC. C. E.—The questions connected with the design of masonry dams, presented for discussion by Mr. Harrison, are of great importance. In this paper the author has indicated the conclusions at which one should arrive with reference to upward water pressure under the base of a dam in designing the profile, namely, that it would depend on the rock on which the dam was to be founded. In some cases there might be considerable upward pressure, while in others little, if any, allowance should be made for such a force.

It might be of interest to examine briefly the evolution of the theory of designing masonry dams. Prior to 1853 such dams were built without any rational consideration of the forces which they had to resist. Some of these structures would be stronger if they could be turned about so as to make their up-stream faces their down-stream sides.

The French engineer, De Sazilly, in his paper on "A Profile-Type of Equal Resistance for Reservoir Walls,"* explained, for the first time, the principles on which the design of a masonry dam should be based. His theory was that, in order to be safe, the structure should have a sufficient factor of safety against overturning and sliding, and that the maximum pressure to which the masonry and foundation were to be exposed should not exceed a safe limit. As far as the last mentioned element of strength is concerned, De Sazilly's ideal type of dam was one in which the maximum pressures in the masonry, at different levels, were exactly equal to the highest allowable amount.

Delocre, Bouvier, Pelletreau, Lévy, and some other French engineers, have written papers, proposing profiles differing somewhat from De Sazilly's type. Professor Rankine added to the requirements established by the French engineers the important condition that the lines of pressure, for reservoir full or empty, should be kept within the center-third of the profile, in order to avoid tension in the masonry; and he also pointed out that the assumed limit of vertical pressure should be lower at the down-stream face than at the up-stream face. The profile designed by Rankine, which was adopted for the Toolsay Dam, near Bombay, 79 ft. high, was bounded by logarithmic curves.

* *Annales des Ponts et Chaussées*, 1853, Vol. II, pp. 191–222.

Mr. Wegmann.

As far as the speaker knows, none of the engineers mentioned took into account an upward pressure under the base of the dam, or ice pressure, and yet, a great many dams have been built according to their types and none has failed, with the exception of one in France, built on a bad foundation, and another in Africa, constructed in a very defective manner.

In 1884 the engineers of the Aqueduct Commission of the City of New York had to design a masonry dam, from 275 to 300 ft. high, which was to be built across the Croton River, near its mouth, to form a large storage reservoir for the water supply of that city. The site originally selected for the proposed dam was at the Old Quaker Bridge, about 4 miles above the mouth of the river, and, because of this location, it was called the Quaker Bridge Dam. The structure was eventually built, in 1892 to 1907, 1¼ miles farther up stream, and was named the New Croton Dam, in contradistinction to the Old Croton Dam, 3 miles farther up stream, built in 1837 to 1842.

The necessary studies and investigations were made under the direction of the late Alphonse Fteley, Past-President, Am. Soc. C. E., Deputy Chief Engineer, and later Chief Engineer, of the Aqueduct Commission, and it was the speaker's privilege to be Mr. Fteley's principal assistant in this matter.

The highest reservoir wall in existence in 1884 was the famous Furens Dam, near Saint Etienne, France, which had a maximum height of about 184 ft. above the foundation. In America there was at that time only one masonry dam, not considering low structures, namely, the Boyd's Corners Dam, built in 1866 to 1873, across the West Branch of the Croton, and having a maximum height of 78 ft. above the foundation.

The formulas for determining the proper profile of a masonry dam, which had been published up to that time (namely, those of De Sazilly, Delocre, Rankine, Pelletreau, De Bauve, etc.), were very complicated and unsatisfactory. They required that the maximum pressures in the dam, at different levels, should be kept at a fixed limit of safety, which was taken by these different engineers at from 6 to 10 tons per sq. ft. If the design of the proposed dam across the Croton River had been based on such data, its down-stream face would have become almost horizontal at a depth of 300 ft. The only practical solution of the problem was to assume a higher limit for the permissible pressure in the masonry and on the foundation, and this was fixed by the engineers of the Aqueduct Commission at 32 000 lb. per sq. ft., or about twice as much as had been considered safe by the French engineers up to that time. What confirmed the designers of the New Croton Dam in the reasonableness of their assumption of maximum pressure was, not only experiments on the crushing strength of masonry, but the

Mr. Wegmann. fact that the Almanza Dam, built in Spain prior to 1586, the oldest masonry dam in existence, was sustaining safely a pressure of about 14 tons per sq. ft.

The possibility of an upward pressure under the base of the dam was considered carefully by the engineers of the Aqueduct Commission and by a Board of Experts consisting of three prominent engineers, Messrs. J. P. Davis, J. J. R. Croes, and W. F. Shunk, and such a force was left out of consideration, in this case, on the ground that with the depth to which the foundation was to be excavated and the care contemplated in closing all fissures and seams in the rock, an upward pressure, if it occurred at all, would in all probability extend over only a very small part of the base.

The Board of Experts advised that the dam be curved in plan, and that it be designed to resist, in addition to the water pressure, an ice thrust of 43 000 lb. per lin. ft. at the highest ice line. The plan submitted by this Board, however, was not adopted, and the dam was built straight in plan, no attention being given to upward pressure or ice thrust.

Mr. Fteley, who had had large experience in the construction of dams, followed the same method in designing two other masonry reservoir walls in the Croton water-shed, namely, the Sodom Dam, 98 ft. high, built in 1888 to 1893, and the Titicus Dam, 135 ft. high, built in 1890 to 1895.

The three dams just mentioned are standing successfully, and do not show the slightest indication of weakness. Thus far, the New Croton Dam is the greatest work of its kind. The Shoshone Dam, in Wyoming, surpasses it in height by about 25 ft., and the Assuan Dam, in Egypt, contains a greater quantity of masonry, owing to its great length; but the former is built across a narrow cañon, and the latter has a maximum height of only about 112 ft. The Olive Bridge Dam, now being built in connection with the Ashokan Reservoir, in Ulster County, for the City of New York, has a maximum height of only 252 ft.

In 1895 John D. Van Buren, M. Am. Soc. C. E., in a paper entitled "Notes on High Masonry Dams,"* advocated that a masonry dam be made strong enough to resist a pressure under its base equal to the full hydrostatic head in the reservoir, and that in northern latitudes the dam should also be able to withstand an ice pressure of about 40 000 lb. per lin. ft. at its highest ice line. With these conditions, he designed the profile shown in Fig. 3,† the width of the base being 352 ft. and the height 250 ft. If these principles had been followed in designing the New Croton Dam, having a maximum height of

* *Transactions*, Am. Soc. C. E., Vol. XXXIV, p. 493.

† Reproduced from Mr. Van Buren's paper.

297 ft. above the foundation, its base would have been more than 400 ft. wide, instead of 206 ft., as actually built. Mr. Wegmann.

According to a letter by G. M. Braune, Assoc. M. Am. Soc. C. E.,* the Marklissa Dam, in Prussia, completed in 1904, was designed to resist a pressure under its base equal to the full hydrostatic pressure in the reservoir, "in order to allay the fears of the citizens living below the dam." In this case the width of the base was about 87% of the height (namely, width of base 37.7 m. for a height of 43 m.). Masonry dams might be built on foundations so porous as to justify the extreme assumption on which the design of the Marklissa Dam was

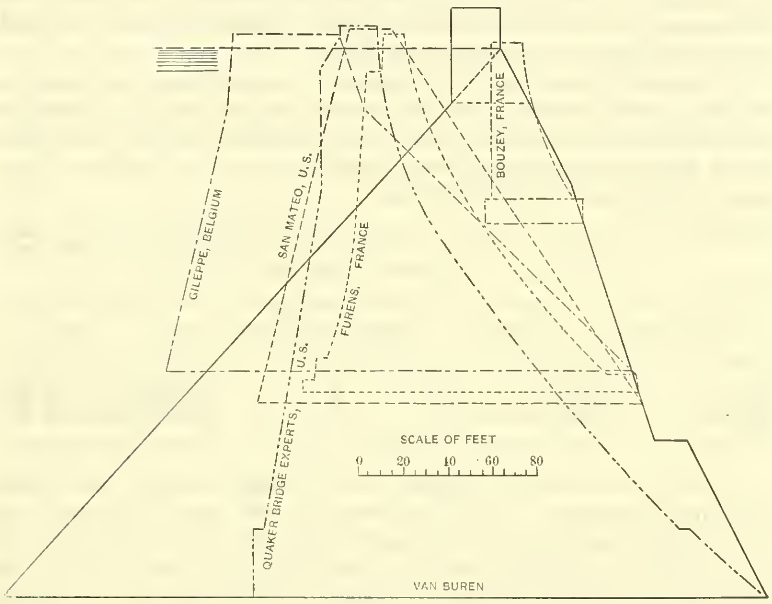


FIG. 3.

based, but such cases are likely to be extremely rare; and, to adopt such a design for an ordinary foundation would be like calculating a bridge for a factor of safety of 10, when experience had shown that 6 was ample.

The Wachusett Dam, built in Massachusetts in 1900 to 1906, was the first American dam in the design of which an upward pressure under the base was taken into account. In this case there was a town of 13 000 inhabitants about half a mile below the site, and failure would have caused a great disaster. Under these circumstances, the engineers adopted a very strong profile, calculated to resist an upward

* *Engineering News*, Nov. 30th. 1911, p. 661.

Mr. Wegmann. pressure under the base equal to two-thirds of the pressure of the water at the up-stream face and diminishing uniformly to zero at the down-stream toe, and, also, an ice pressure of 47 000 lb. per lin. ft. of dam at the highest ice line.

In the Cross River and Croton Falls Dams, built recently in the Croton water-shed, the designs were based on similar conditions, an upward pressure under the base being calculated in the same manner as for the Wachusett Dam, but the ice pressure per linear foot was taken at only 24 000 lb. for the Cross River Dam, and at 30 000 lb. for the Croton Falls Dam. These structures are near the New Croton Reservoir, no habitations being directly below them, and under these circumstances it is a debatable question whether the profiles should have been made as strong as they were.

There are three cases of failures of masonry dams on record in which upward pressure under the base of the structure, for which no allowance had been made in the design, doubtless played a large part. Some particulars regarding these dams are given in Table 1. They were all built on poor foundations.

TABLE 1.—Masonry Dams in Which the Failure was Largely Due to Upward Pressure.

Name of dam.	Location.	Maximum height.	Built.	Failed.
Bouzey	France	72 ft.	1878–1881	April 27th, 1895.
Austin	Texas	68 "	1891–1892	April 7th, 1900.
Austin	Pennsylvania	50 "	1909–1910	Sept. 30th, 1911.

The Bouzey Dam was founded on red sandstone, which was fissured and quite permeable. The foundation for the dam itself was only excavated to fairly good bottom, and not to solid rock. A cut-off wall, 2 m. thick, was built at the up-stream face, from solid rock to the river bed. The first time the reservoir was filled, a portion of the dam, about 440 ft. long, was shoved forward so as to form a curve, convex down stream, having a versed sine of 1.1 ft. The dam was reinforced by building, on its down-stream side, an abutment which was connected with the dam by an inclined wall toothed into the dam. This, however, proved to be insufficient, and a length of about 590 ft. of the dam was overturned as soon as the reservoir was filled again.

The dam across the Colorado River at Austin, Tex., was built on a foundation which was very poor in places, part of it being on a fault, 75 ft. wide, filled with adobe with an occasional streak of red clay. The foundation trench was not excavated deep enough, and the protection against erosion on the down-stream side was insufficient.

The failure of the dam at Austin, Pa., is so recent that it does not need much description here. The dam was founded on sandstone, underlaid by shale having fissures filled with clay, sand, and gravel. Mr. Wegmann.

In each of these three cases there was, doubtless, considerable upward pressure under the base, and each profile should have been designed to give the dam sufficient strength to resist such a force.

From the foregoing facts, the speaker draws the conclusion that uplift under a dam may vary from nothing to a considerable force. How much upward pressure should be assumed in any given case is purely a matter of judgment, and in each case it is advisable for the engineer to consult some experienced geologist.

Some water will find its way through the best built masonry dam and will appear as damp spots on its down-stream face. The question has been raised as to whether such water might not exert some upward pressure in the masonry. Unless some provision for expansion and contraction is made, cracks are sure to occur in the masonry. If these cracks are nearly vertical, little or no upward pressure will be caused by water from the reservoir filling them; but if the expansion cracks are about horizontal, upward pressure will, doubtless, occur in the masonry, its magnitude depending on the depth to which the cracks extend into the wall.

When uplift under the base of a dam is considered in designing the profile, provision is usually made for it at any level above the base, as the same profile is generally used for the highest parts of the dam and for the low parts at the two ends. For this reason the profile would be designed so that, when the base was taken at any level below the top of the dam, the masonry above the base would have ample strength to resist the thrust of the water on its up-stream face and, also, the assumed upward pressure under its base.

The question as to how much should be allowed in northern latitudes for ice pressure against a masonry dam is difficult to answer with the information now available. Ice, when confined, as between two bridge piers, doubtless exerts great pressure in expanding, but when this ice is in a reservoir having sloping banks, the dam being usually in a narrow gorge, it is doubtful whether it would exert much pressure against the dam. Ice forms every winter against weak mill-dams, and yet we do not hear that they are ruptured by it. It would be interesting if experiments were made to measure the expansive force of ice, but this is difficult to accomplish on a scale sufficiently large to be of value. One way to make such a test would be to let ice form directly against flash-boards, first determining their resistance to overturning and shearing, and then noticing whether they were forced out.

It is certain that, if the great uplift and ice pressure which some engineers recommend had been realized, many masonry dams, now standing successfully, would have been ruptured long ago.

Mr. Wegmann. The speaker has tested by calculation the strength of some American and foreign dams to withstand a full upward pressure under their bases, and an ice pressure of 25 tons per lin. ft., an assumption which some engineers think should be made in designing, and has found the lines of pressure, reservoir full, to fall for a great part outside of the profile. In such a case a dam would be overturned unless this were prevented by the tensile strength of the masonry. The speaker is satisfied that such a condition does not exist in the dams to which he refers. As far as the speaker knows, only one case is on record in which the expansion of ice was probably the primary cause of the failure of a dam; this was the partial failure of such a structure at Minneapolis, Minn.* In this case, however, there were special conditions.

In conclusion, it may be stated that, if ample security against failure were all that the engineer had to seek in designing a dam, his task would be very easy. He might follow the simple rule of making the width of the dam, at any level, equal to one and one-half times the height, and feel sure that nothing short of an earthquake would cause rupture. There remains, however, the consideration of cost, and the best engineer is he who obtains the required security and architectural appearance of the dam, at the least expense. Engineers designing works to be built for great cities are somewhat relieved from the necessity of counting the costs of the projected work, as they may feel quite confident that whatever plan they may recommend will be carried out. It is very different, however, when private capital furnishes the means for building such works. The engineer who combines, with his theoretical knowledge, sound judgment and courage to follow his convictions, is the ideal type which all should strive to follow.

Mr. Gregory. CHARLES E. GREGORY, ASSOC. M. AM. SOC. C. E.—The influence of the upward pressure of water and of ice thrust on the stability of masonry dams, together with the actual internal distribution of stress in very large masses of masonry, are probably the most indefinite factors in the design of such structures.

The speaker, as Designing Engineer, under the direction of J. Waldo Smith, M. Am. Soc. C. E., Chief Engineer, and Alfred D. Flinn, M. Am. Soc. C. E., Department Engineer, of the New York Board of Water Supply, has designed two high masonry dams. The first, Olive Bridge Dam, helps to form Ashokan Reservoir, about 13 miles west of Kingston, N. Y; the second, Kensico Dam, forms Kensico Reservoir, at Valhalla, N. Y. Fig. 4 is a cross-section of Olive Bridge Dam, and on Plate V there is a cross-section of Kensico Dam, as well as brief statements of the assumptions made in its

* *Engineering News*, May 11th, 1899; and *Engineering Record*, May 13th, 1899.

design, pressure lines, diagrams of maximum compressive stresses, vertical and horizontal shearing stresses, etc. On each illustration there is a "theoretical section" obtained from the assumptions stated on Plate V and the further condition that there should be no stress at the up-stream edge for full water and ice load and none at the down-stream edge for reservoir empty. These sections were worked out by a direct analytical method, developed under the speaker's direction, by O. L. Brodie, Assoc. M. Am. Soc. C. E., for the Olive Bridge Dam, and described by him in his discussion of this paper. Mr. Gregory.

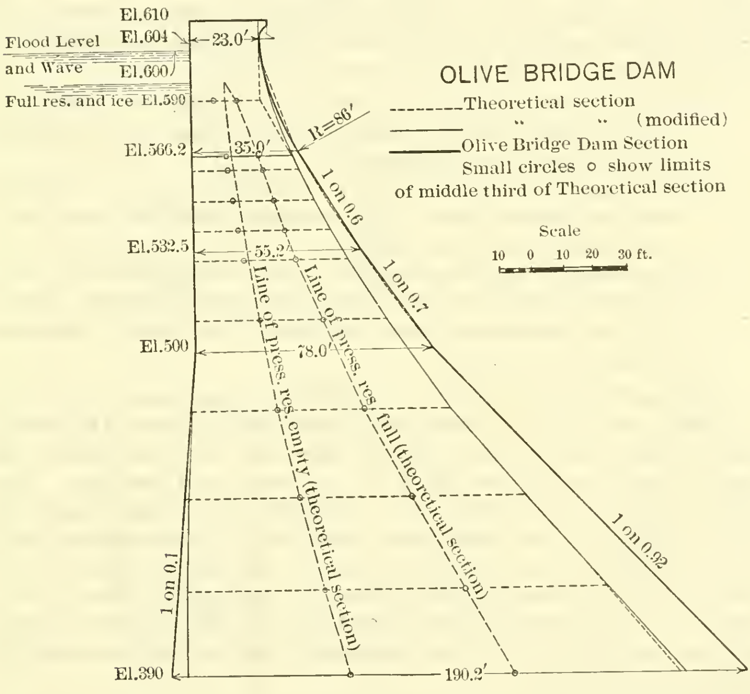


Fig. 4.

The bulge in the theoretical section just below the flow line indicates the influence of the ice pressure. The ice thrust has a greater influence on the thickness of the dam than the flood-water level, down to about 110 ft. below the flow line in the Olive Bridge Dam, at which point the 10-ft. flood level begins to require a wider base; and in Kensico Dam, with a flood level of 5 ft., the change is about 210 ft. below, indicating that, for dams of moderate height, ice pressure has a great influence.

Mr. Gregory.

Kensico Dam at its greatest height is expected to extend more than 130 ft. below the surface of the ground. In designing this section, the effect of the earth and water pressure against both sides of the dam, and their influence on the uplift under the bottom, were taken into account, as indicated on Plate V. The effect of the earth pressures on the down-stream slope is to increase the unit compressive stress in the masonry, but to decrease the overturning moment.

The weight of the masonry was corrected for the galleries and wells, and was found to be very nearly two and one-third times the weight of water, and this value was adopted. For these dams, both analytical and graphical methods of determining stresses were used for the adopted sections, and these were made greater than the "theoretical" sections for the following reasons:

(1) Both dams are just above very populous districts, and their failure would be such a calamity that a liberal factor of safety was thought to be imperative;

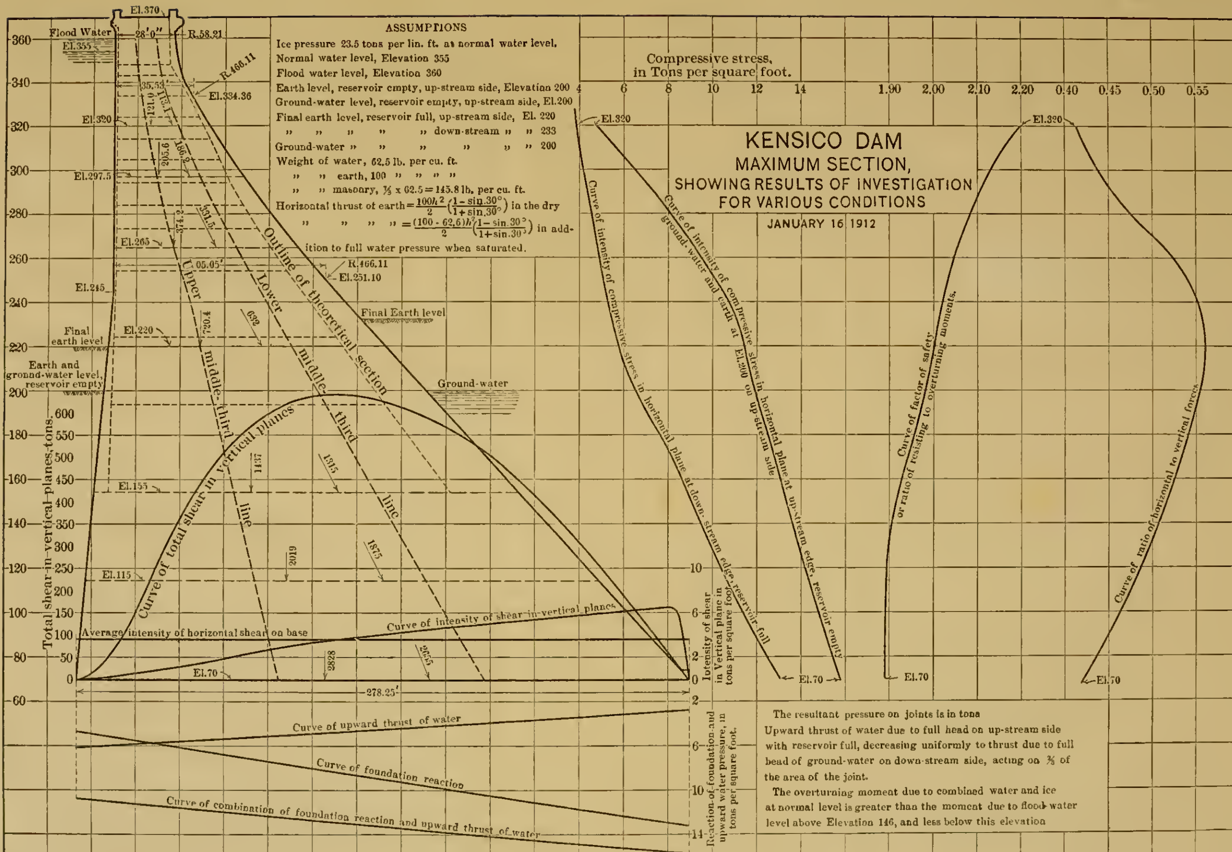
(2) The face of the dam, being of a different kind of masonry than the interior, may in time separate from the main body, and become practically useless;

(3) Interior, vertical, longitudinal cracks may weaken the dam by separating it into two or three parts, which, to some extent, will act separately, thus destroying the vertical shearing strength of the section.

Both these dams are built on stratified rock, and each is to have a cut-off of considerable depth along its up-stream face. The stratification planes at the Olive Bridge Dam are nearly horizontal; at Kensico, they are inclined about 45° to the axis of the dam. The ordinary masonry dam built on a stratified rock is most likely to fail by sliding, if the down-stream toe does not abut against solid rock. In addition to the friction between planes due to the weight of the dam, which is ample, sliding along a stratification plane is prevented in these dams by the depth to which the masonry is built into the rock, the toe abutting horizontally against an approximately vertical face of rock.

As Mr. Harrison states, upward pressure may be due to water getting into the foundation or into the dam itself. The foundation conditions at the Olive Bridge Dam are quite similar to his Case 2, and those for the Kensico Dam are in effect like Case 3. The cut-off trench in each case undoubtedly reduces the leakage, but might not cut down the pressure materially, as experience has shown it to be very difficult to build a concrete wall of the lengths required without shrinkage and temperature cracks.

The part of the base over which the pressure is active cannot be the entire area, unless the dam is actually floating. The total pres-


KENSICO DAM
MAXIMUM SECTION,
SHOWING RESULTS OF INVESTIGATION
FOR VARIOUS CONDITIONS
JANUARY 16 1912
ASSUMPTIONS
Ice pressure 23.5 tons per lin. ft. at normal water level.
Normal water level, Elevation 355
Flood water level, Elevation 360
Earth level, reservoir empty, up-stream side, Elevation 200
Ground-water level, reservoir empty, up-stream side, El. 200
Final earth level, reservoir full, up-stream side, El. 220
" " " " " down-stream " " 233
Ground-water " " " " " 200
Weight of water, 62.5 lb. per cu. ft.
" " earth, 100 " " " "
" " masonry, 7/3 x 62.5 = 145.8 lb. per cu. ft.
Horizontal thrust of earth = $\frac{100h^2}{2}\left(\frac{1-\sin 30^\circ}{1+\sin 30^\circ}\right)$ in the dry
" " " " = $\frac{(100-62.6)h^2}{2}\left(\frac{1-\sin 30^\circ}{1+\sin 30^\circ}\right)$ in addition to full water pressure when saturated.
Compressive stress, in Tons per square foot.
Flood Water
El. 370
El. 355
28'0"
R.58.21
R.466.11
El. 334.36
El. 320
El. 297.5
El. 265
El. 251.10
El. 245
El. 220
El. 155
El. 115
El. 70
278.25'
Final earth level
Earth and ground-water level, reservoir empty
Final Earth level
Ground-water
Upper middle third line
Lower middle third line
Outline of theoretical section
Curve of total shear in vertical planes
Total shear in vertical planes, tons.
Average intensity of horizontal shear on base
Curve of intensity of shear in vertical planes
Intensity of shear in Vertical plane in tons per square foot
Curve of upward thrust of water
Curve of foundation reaction
Curve of combination of foundation reaction and upward thrust of water
Reaction of foundation and upward water pressure, in tons per square foot.
Curve of intensity of compressive stress in horizontal plane at down-stream edge, reservoir full
Curve of intensity of compressive stress in horizontal plane at up-stream edge, reservoir empty
ground-water and earth at El.200 on up-stream side
Curve of factor of safety or ratio of resisting to overturning moments.
Curve of ratio of horizontal to vertical forces
The resultant pressure on joints is in tons
Upward thrust of water due to full head on up-stream side with reservoir full, decreasing uniformly to thrust due to full head of ground-water on down-stream side, acting on 2/3 of the area of the joint.
The overturning moment due to combined water and ice at normal level is greater than the moment due to flood water level above Elevation 146, and less below this elevation

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sure on the parts of the joint in contact must equal the difference between the weight of the dam and the uplift of the water, and must be within the crushing strength of the parts in direct contact, or say, not more than 500 lb. per sq. in. The assumption that only one-third of the rock faces are in direct contact gives pressures as great as probably occur, and leads to the conservative assumption of upward water pressure over two-thirds of the base, and varying according to the resistance losses in passing through the rock or masonry. Where upward pressure must be provided for at the base, very little, if any, economy is possible in designing the portions above without allowing for it.

In all but the portions below the creek bed, water pressures in the masonry are controlled by vertical drainage wells about 10 ft. from the up-stream face and 12 ft. apart.

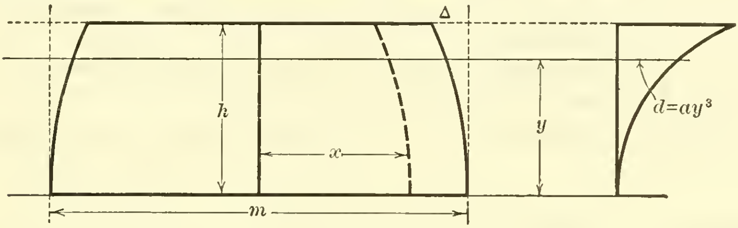

FIG. 5.

Water may enter the masonry through the small pores in the mortar, where no cracks or joints exist; but the uplift which it exerts cannot be applied over a greater part of the joint than is indicated by the fraction obtained from dividing the percentage of water which the masonry will absorb by the percentage of voids in the mass before setting. Water, however, is more likely to enter the body of a cyclopean masonry or concrete dam, so as to cause a material uplift, through defective construction joints, as in the dam at Austin, Pa., or through temperature cracks. Large bodies of Portland cement concrete which have been deposited rapidly during a summer season maintain a temperature of more than 100° Fahr., for a long time, due to chemical action while the cement is setting. In winter the low temperature of the atmosphere reduces the temperature of the outside skin to a point 60° and possibly 100° below that which is maintained by chemical action several feet from the surface, and the consequent contraction of the outside causes cracks which seek to follow weak lines in any direction in which they may occur.

Such cracks will occur in first-class masonry as well as in that of inferior quality, and will probably be larger and farther apart the stronger the concrete. A general formula for indicating the spacing

Mr. Gregory.

of temperature cracks, used by F. F. Moore, M. Am. Soc. C. E., may be applied here, and would be as follows:

In Fig. 5,

Let $m =$ the distance between temperature cracks;

$h =$ the height of wall, or the distance from the outside to the point where the temperature is nearly constant;

$x =$ any distance less than $\frac{m}{2}$;

$\varDelta =$ the distance the outside edge would contract if unrestrained, due to temperature changes in the distance $\dfrac{\frac{m}{2} + x}{2}$;

$d =$ degrees, Fahrenheit;

$c =$ coefficient of contraction in masonry due to temperature changes;

$f =$ the strength of concrete, in pounds per square inch;

$E =$ the modulus of elasticity of masonry.

The cantilever bounded by the vertical lines, at distances x and $\frac{m}{2}$ from the center, has deflected at its end $\triangle = \dfrac{\frac{m}{2} + x}{2} c\, d.$

If d is uniform for the depth, h, and for no crack to occur at the center,

$$\triangle = c\, d \frac{\left(\frac{m}{2} + x\right)}{2} = \frac{f\, h^4}{8\, E \dfrac{\left(\frac{m}{2} - x\right)^3}{12}}$$

and f is the maximum when $x = 0$, and

$$m = 2\, h \sqrt[4]{\frac{3\, f}{c\, d\, E}}, \text{ or } f = \frac{E\, c\, d}{3} \left(\frac{m}{2\, h}\right)^4.$$

Assuming that d varies with the distance from the base, y, and that $d = a\, y^3$ and the maximum $d = a\, h^3 = d'$, where a is a constant for each value assumed for the maximum d, and that $f = c\, E\, d = c\, E\, a\, y^3$, then when

$$y = h,\ \triangle = \frac{c\, E\, a\, h^7}{16.8\, E \dfrac{\left(\frac{m}{2}\right)^3}{12}} = \frac{c\, d' \left(\frac{m}{2}\right)}{2}, \text{ and } m = 2\, h \sqrt[4]{\frac{10\, f}{7\, c\, d'\, E}}$$

$$\text{or } f = \frac{7}{10}\, c\, d'\, E \left(\frac{m}{2\, h}\right)^4$$

During the winter of 1910-11 numerous cracks appeared in the masonry of the partly completed Olive Bridge Dam. One of these was horizontal, and many others were vertical longitudinal ones. Somewhat similar cracks, it is said, have appeared in other dams, under similar circumstances. Such cracking of masonry leads to a very brief discussion of its effect on the internal strength of a dam. Mr. Gregory.

Vertical cracks normal to the axis of the dam are prevented by expansion joints (or, better named, contraction joints) at proper intervals. The distance between these joints is about one-half of the maximum thickness of the dam, and consequently a longitudinal crack is more apt to occur from joint to joint than a crack across the dam between joints. The seriousness of these cracks depends on their extent and width, particularly whether they extend up through the down-stream face, whether the sides are flat planes or are very irregular and toothed into each other, and on the extent to which water under pressure has access to them. The speaker will not give any results of computations based on these various assumptions, but, from a glance at the stress diagrams for Kensico Dam, it is evident that great changes in stress would result from weakening or destroying the vertical shearing strength near the middle or down-stream part of the dam.

As Mr. Harrison has stated, the greatest pressures due to ice are caused by the expansion (under a higher temperature) of ice formed at a low temperature. The force which might be exerted by the impact of ice floes is probably very much less than that due to expansion, as there is no probability that, in an ordinary storage reservoir, ice floes could attain sufficient velocity to cause greater pressure. Such pressures would also be exerted only over small areas at a time, because there would always be projecting points which would be broken off one at a time, thus gradually bringing the floe to rest. It was assumed that clear block ice, 1 ft. thick, might form and, under the conditions described, might expand so as to exert nearly its full crushing strength of about 47 000 lb. per lin. ft. of dam. This figure was used for the Wachusett Dam, and 43 000 lb. was recommended by the Board of Experts for the Quaker Bridge Dam, while 24 000 and 30 000 lb. per lin. ft., respectively, were assumed for the recent smaller and less important dams in the Croton water-shed at Cross River and Croton Falls. It is undoubtedly a fact that many dams have been built without making allowance for ice thrust in their design; but many such dams are made strong enough to resist considerable ice thrust by assuming a super-elevation for floods and considerable top widths for roadways. It is also true that, while the ice thrust conditions assumed for the large dams of the Board of Water Supply of New York are entirely possible, they actually occur very rarely, and, in many dams, probably have not yet obtained. Just the proper combination of

Mr. Gregory. temperature and a stationary water surface at the maximum elevation for a sufficient time may occur, but not frequently.

Mr. Brodie. Orrin L. Brodie, Assoc. M. Am. Soc. C. E.—The speaker was Mr. Gregory's assistant in connection with the two large dams which he mentions. So much has been said in the discussion of this paper that there seems to be little left. However, one feature, which appears to have been overlooked, relates to one of the speaker's early experiences in connection with some small retaining wall work of which he was in charge, and in which the contractor and builder endeavored to preserve the continuity of the structure from day to day. Observations during subsequent seasons showed that although the exposed surfaces or faces of these retaining walls had been left smooth, due to the finish, cracks developed later which were identified at once as being along the horizontal planes where work had been discontinued and started again.

These walls were subject to surcharge. The drainage from the ground behind them was considerable, and there was very noticeable seepage through these cracks, therefore, it is suggested that perhaps during the building of a large structure like a masonry dam, where the work goes on from season to season, horizontal planes of cleavage may occur, and these will admit water and thereby cause uplift.

The usual method of investigating cross-sections is to run lines of resistance through them, either graphically or analytically, under various assumed conditions; but, for a change, the speaker will consider several sections, each designed for an imposed set of conditions. It will of necessity be an academic consideration, but perhaps may be of interest. Further to fix the ideas, the conditions assumed as to general height of dam and water retained are similar to those of the dam at Austin, Pa., because it is presumed that most engineers are more or less familiar with the cross-section of that structure. It is not the intention, however, to enter into any discussion of its failure, but simply to use it as a type for illustration.

Five profiles or cross-sections are presented in Fig. 6, one of which is that of the Austin Dam; the other four are designated by the letters, *A, B, C,* and *D*. The asterisk in Table 2 signifies that *B* and *C* are subject to the same conditions of loading, the consideration of *C* being only incidental.

The conditions of loading for *A* will be ice pressure, uplift, and horizontal water pressure. The assumptions, with the exception of the ice pressure, will be similar to those used by the Board of Water Supply in its studies. The speaker has reduced the ice pressure to about one-half of that used by the Board of Water Supply, because the dam is rather low, and the excessive pressure would require a much greater top than is necessary, both in thickness and super-elevation, and, besides, only illustration and comparison are desired. Sections

B and *C* are designed for uplift and horizontal water pressure only. Section *D* will provide for only the horizontal water pressure. Mr. Brodie.

The uplift intensity is assumed as varying uniformly from a maximum at the heel to zero at the toe, and, as stated by Mr. Gregory, the

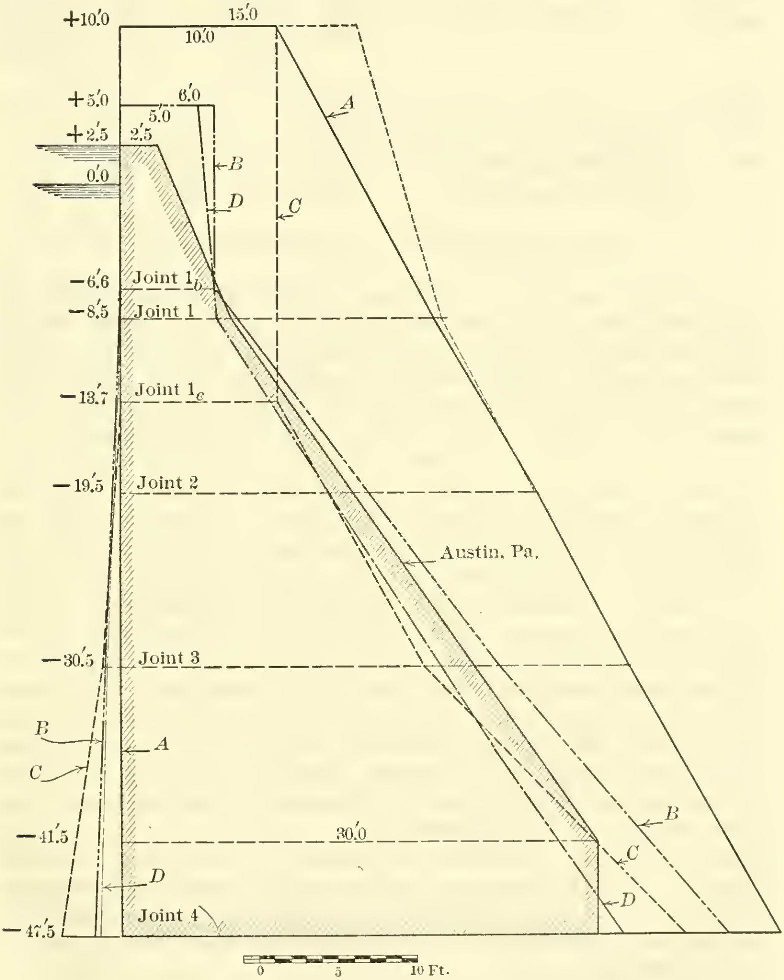

Fig. 6.

uplift is considered as acting over only a portion of the area of the joint. This is cared for by assuming only a portion of the full hydrostatic head as acting at the up-stream end of the joint considered. Two-thirds of the full up-stream head is used.

Mr. Brodie.

TABLE 2.—DIMENSIONS, CONDITIONS, ETC., FOR FIVE CROSS-SECTIONS OF DAMS.

Cross-sections.	Conditions of loading.	Top width, in feet.	Super-elevation above normal reservoir surface, in feet.	Base, in feet.	Area, in square feet.	Percentage of excess of area over the area of *D*.
Austin, Pa.	....	2.5	2.5	30	840±	0±
A........	Ice, uplift, and horizontal water thrust............................	10.0	10.0	41.5	1 475	76
*B**.......	Uplift and horizontal water thrust.	6.0	5.0	40.0	992	19
*C**........	" " " " "	10.0	10.0	39.5	999	20
D........	Horizontal water thrust...........	5.0	5.0	33.1	836	0

* *B* and *C* are subjected to the same conditions of loading.

The speaker agrees that a distinction should be drawn between the uplift conditions which may be encountered in the foundations and those higher up in the dam, and suggests that the foregoing assumptions in regard to uplift may be modified for special cases. For example, a trapezoidal (instead of a triangular) distribution of intensity due to uplift may be found advisable for a foundation. In cases where intercepting drainage wells are provided in the body of the masonry, as for the Olive Bridge and Kensico Dams, it would be reasonable to assume a triangular disposition of intensities, with the maximum at the heel as before, but running out to zero at or a little beyond the line of wells. In the former case, the total pressure would be increased but its lever arm would tend to be diminished. In the latter case, the pressure would be diminished but the lever arm increased. The effect on a cross-section design, or on a line of pressure for a given cross-section, would have to be worked out for any particular case. For such a structure as the Kensico Dam, the foregoing changes from the ordinary triangular assumption, while modifying the numerical results as to "factors" against overturning and as to resulting pressure intensities on the various joints and the foundation, did not modify the final cross-section.

In a discussion regarding ice pressures, before the Canadian Society of Civil Engineers in December, 1891, agreement seems to have been reached on two points: That thrust from ice less than 3 in. thick can be disregarded, and that the thrust can safely be taken at the crushing strength of ice.

As the compressive strength has been recorded† as ranging from 100 to 1 000 lb. per sq. in., and as there are other variable factors affecting thrust which have been pointed out in the discussions of this paper, the uncertainties of the problem are at once apparent. Addi-

† *Engineering News*, January 12th, 1893, p. 41; and also April 5th, 1894, p. 285.

tions to the meager data now available should be made by those who have had occasion to observe ice thrust phenomena. Mr. Brodie.

The value assumed in this discussion is 21 500 lb. per lin. ft. of dam. For ice about 3 in. thick, this would be equivalent to about 600 lb. per sq. in. The value used by the Board of Water Supply was 47 000 lb. per lin. ft., and is equivalent to about 650 lb. per sq. in. for ice 6 in. thick.

The condition that the lines of pressure, for reservoir full or empty, must not pass outside the middle-third limit of any joint is introduced into the equations; likewise, the condition that the maximum working pressure on any horizontal joint must never exceed certain prescribed limits. The condition limiting the inclination of the resultant is tested afterward.

The maximum working pressure condition does not need investigation here, because, for a dam of the height used in this discussion, the resulting pressure intensities are low. The line of pressure, therefore, is at the down-stream end of the middle third of each joint, reservoir filled, for each of the cross-sections, *A, B, C,* and *D;* hence, they may be considered minimum sections for their respective conditions. The fluctuation of reservoir level is taken at 2.5 ft. As ice is generally broken up during flood, the ice thrust is taken at the normal reservoir level.

The dimensions of the Austin Dam, together with the density of its masonry, 140 lb. per cu. ft., were first obtained by the speaker through the courtesy of Alfred D. Flinn, M. Am. Soc. C. E., Headquarters Department Engineer, Board of Water Supply. This weight was used in calculating the different cross-sections.

It is usual to assume the section and compute the lines of pressure, or work them out graphically. The procedure followed in the Board of Water Supply was analytic, checked by the graphic method, wherein minimum sections were calculated directly as is done here, and then chosen ones were investigated. The formulas for design, which were adapted by the speaker from those which Mr. Wegmann early developed, contain conditions of uplift and ice pressure. It is not proposed to give all these formulas here; but the fundamental method of analysis will be indicated for those who may be unfamiliar with work of this kind. Formulas for investigation were also developed, and included other forces than those mentioned here.

In Fig. 7, *a-b,* or *l,* may represent the length of the dam base for a unit length of dam; and it may be divided into three parts, *y, v,* and *u; a* is the up-stream end of the joint. Let *W* represent the position of the weight of the mass of masonry acting through its center of gravity, horizontally distant *y* from *a.* Then the center of pressure when the dam is subject to the entire loading, both water pressure and uplift, or whatever is wished to be assumed, will be distant *v* from *W,* Mr. Brodie. and the remaining distance or segment of the base considered will be called u. We then have the equation, $l = y + v + u$. If moments be taken about e, the center of pressure of the resultant when the dam is subject to the loading, and M be that moment, tending to overturn the mass of the dam, M divided by W will equal v.

We may substitute $\frac{M}{W}$ for v, in the equation for l, and then whatever may be the overturning moment due to any forces considered, and whatever the conditions due to the shape of the structure (that is, y may or may not be an assumed condition), and whatever limit or condition that is wished to be placed upon u (that is, the distance of

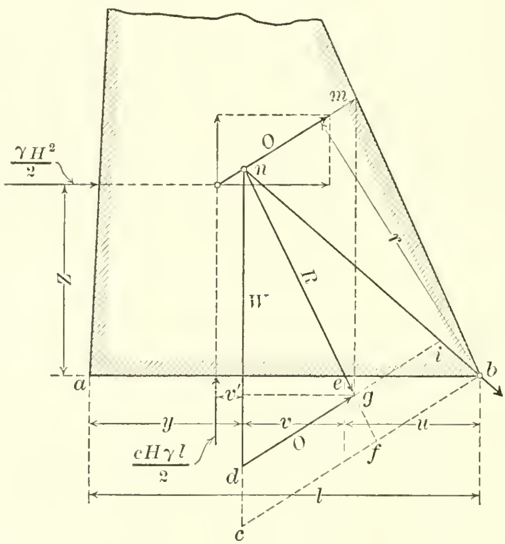

Fig. 7.

the center of pressure from the down-stream end of the joint), there will result an equation involving l. By substituting imposed conditions for y and u, and the loading, and then reducing, a quadratic equation is obtained, and this, being in terms of l, will then enable us to solve the base directly. Therefore, it is evident that, by introducing such conditions, there could be developed a series of appropriate equations for design. The governing conditions vary, as one goes from the top of a dam downward, so that these conditions have to be introduced successively; therefore, one simple working equation cannot be formed for the entire cross-section. That is all the speaker wishes to say as to the analysis at this stage, deeming it a sufficient indication of the methods of determining the sections which will now be presented.

Mr. Brodie.

First, consider Section *A*, Fig. 6, which has a 10-ft. top, a 10-ft. super-elevation from the normal reservoir level, and a 41½-ft. base. It resulted from the conditions of ice at the normal reservoir level and the resultant center of pressure at the middle-third point at each joint.

Second, Section *B* resulted in a 6-ft. top width and a 5-ft. super-elevation.

Third, Section *D* has a 5-ft. top width and a 5-ft. super-elevation.

Thus, the super-elevations are, respectively, 10 ft., 5 ft., and 5 ft.; with the bases, areas, etc., as recorded in Table 2.

The dam at Austin, Pa., was 2½ ft. wide on the top, with a 2½-ft. super-elevation; the down-stream face sloped down to 7 ft. from the up-stream face at 11 ft. below the top, and then battered off to 30 ft. at 44 ft. below the top, from which level it continued vertically down 6 ft. farther; its back was vertical for the full 50 ft.

The points wished to be brought out are these: By applying direct analytic solution methods, it was found that Section *A* had to have a super-elevation of 10 ft. The top had to be at least 10 ft. wide; by making it 15 ft., the line of the down-stream face, worked out, joined the face of the structure having a 10-ft. top, about 17 ft. below the water level, and continued thence with about the same slope as for the cross-section with the 10-ft. top, showing that there was not much advantage in increasing the width; but there is an advantage in having a super-elevation, especially for the ice pressure.

One would naturally infer that the top would be most affected by that thrust, and this is shown to be so by comparing Sections *A* and *B*, as determined. It will be recalled that Section *B* was calculated without ice-thrust conditions, but with uplift and the horizontal thrust of water on the back.

Section *B* required a lower top, in order to obtain a regular down-stream face. It will be noted, however, that this section differs from *A* in that the up-stream face of the latter is vertical. The series of batters forming this face for Section *B* resulted directly from the necessary application of the formulas containing the middle-third limit condition for reservoir empty, in this case, which did not govern in the case of Section *A*.

Section *C* is subjected to the same conditions as *B*, but with this difference: the super-elevation resulted in the vertical portion of the down-stream face being continued to a lower level than that of *B*; however, the effect of that vertical section on the resulting base lengths below was to throw the lower part of the cross-section well within that of *B* on the down-stream side and then throw it out, lower down, toward *B* and well without the up-stream surface of Section *B*.

As the cross-section is being calculated, proceeding from the top, with only uplift and the horizontal force due to the retained water

Mr. Brodie. acting, it will be found that the resulting cross-section, while more slender at the top, will gradually extend down stream, requiring longer bases, as the determinations continue downward, than were required for the ice-pressure design. This means that at the upper part of the dam the ice thrust governs. (It should be said that, where the ice is disregarded, the head due to the reservoir at flood level is taken for the determination of Sections *B* and *C*.) It is evident, by referring to Fig. 6, that a point would be reached, if the design of the several cross-sections were continued downward, where the profile of either *B* or *C* would come out, crossing that of *A*, and give greater bases than Section *A*, at the same elevations. This shows the importance of designing for ice thrust in low dams, while flood-level conditions govern in high masonry dams.

In designing a high masonry dam, the procedure would be to start with the ice thrust condition and design a section between 80 and 100 ft. high (according to the usual assumptions and conditions, this inter-section of "ice" design and "flood level" design occurs at about 100 ft. from water level), and then start from the top again with the flood level conditions, and go down until a base of the "flood level" section becomes greater than that of the "ice thrust" design at the same elevation; then the remainder of the design is continued with the flood conditions, but with the ice level cross-section superimposed.

The areas of these various sections work out as follows: Section *A*, 1 475 sq. ft.; Section *B*, 992 sq. ft.; and Section *D*, 836 sq. ft.

The cross-section of the Austin Dam worked out to about 840 sq. ft., therefore, it will be seen that Section *D*, which had to meet only the horizontal thrust of the water retained, was practically equal to that of the Austin Dam. Section *A* was some 76% greater than Section *D*, which was used as a basis of comparison, while the uplift as assumed and the horizontal thrust of the water gave Section *B* some 19% greater area than *D*. Thus the economy of the provision for the different designs would be about 19% and 76%, for uplift alone, and for ice and uplift, respectively. This is with a moderate ice assumption.

Concerning Section *A*, another point of interest is that, with this ice design and with uplift, the angle of the resultant (the calculated angle that the resultant makes with the vertical) is only 28 degrees. The speaker believes that M. Lévy recognized 30°, and Professor Rankine 36°, as being the maximum limit. On the other hand, Section *B* had a resultant which made an angle of nearly 38° with the vertical, showing that this direct design for the middle-third condition, with horizontal water thrust and with uplift obtaining, proves unsatisfactory for the base at a level of 50 ft. below the top of the dam; for, while proving adequate for conditions against overturning and maximum pressures, it does not meet the requirements imposed against sliding on the base. Mr. Brodie.

At this point some general observations on the subject of "the factor against overturning" will be made.

There is a difference between the values obtained for the so-called "factor of safety" against overturning, according to whether a ratio of one set of moments, or of another, is taken when uplift is assumed. The first set of moments compared would be the "resisting moment" and the moment of forces tending to overturn the structure. On the other hand, the ratio of the resultant moment of the vertical to the resultant moment of the horizontal components of the forces could be taken.

By referring again to Fig. 7, it will be seen that, for example, the horizontal water pressure on the back, $\frac{\gamma H^2}{2}$, the uplift, $\frac{c\, H \gamma\, l}{2}$, and the weight of the masonry, W, constitute the forces, with the reaction (not shown) considered as acting on the dam. (See *"Nomenclature."*)

The forces tending to overturn the dam about the toe at b are $\frac{\gamma H^2}{2}$ and $\frac{c\, H \gamma\, l}{2}$, the resultant of which is denoted in magnitude and direction by the line, O, at the normal distance, r, from b. The force resisting this tendency is W, the line of action of which is normally distant $(u + v)$ from b. If $n\, m$ denotes the force, O, in magnitude and direction, and $n\, d$, the force, W, then $n\, g$ will represent, in magnitude and direction, the force, R, or the final resultant of all the forces, which is opposed by the reaction (for equilibrium to be assured) at the point, e, distant u from b.

The resisting moment about b, then, is $W\ (u + v) = M_0$ while the overturning moment about the same point is

$$r\, O = M^1 = \frac{\gamma\, H^2}{2} \times z + \frac{c\ H\gamma\ l}{2}\ (u + v + v^1) = M_1 + M_2.$$

Whence, for the ratio of "resisting moment to overturning moment" there may be written:

$$\frac{M_0}{M^1} = \frac{M_0}{M_1 + M_2} \ldots\ldots\ldots\ldots\ldots\ldots (a)$$

and for the ratio of the "resultant moment of the vertical components" to the "resultant moment of the horizontal components" of the forces there follows:

$$\frac{M_0 - M_2}{M_1} \ldots\ldots\ldots\ldots\ldots\ldots (b)$$

These two expressions for the "factor" evidently become equal to each other only when $M_2 = 0$, or when uplift is ignored; also, when the factor of safety is equal to unity (when $M_0 - M_2 = M_1$), or at the point of overturning. Therefore, the usual expression, $\frac{u + v}{v}$, will

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To consider the foregoing discussion for the purpose of developing a graphic treatment for determining the value in either case, it must be remembered that, for the dam to be on the point of overturning, the ratio of the two moments, M_0 and M^1 of Equation a, must equal unity, or $M_0 = M_1 + M_2$, and the line of action, R, in Fig. 7, must pass through b. Inasmuch as W is constant, one or all of the other forces may at this stage be considered variable, in order to bring about the above supposititious condition. According to Equation a the distance, r, is constant, and the water pressure on the back and the uplift, therefore, are supposed to be proportionately increased to fulfill this condition of bringing the line of action, R, through b. This seems reasonable from the fact that the horizontal water thrust cannot be considered to increase without a corresponding increase in the uplift. Therefore, the condition necessary to bring the resultant, R, through b instead of through e, where it actually falls, is that $O = d\ g$ be increased to $d\ i$, in Fig. 7. If $b\ c$ be drawn through b, parallel to O (and, therefore, to $d\ g$), the ratio sought follows from the similarity of the triangles, $n\ i\ d$ and $n\ b\ c$, or, the factor of safety, with respect to resisting moment and overturning moment, is equal to the ratio, $\frac{d\ i}{d\ g} = \frac{c\ b}{c\ f}$.

The foregoing conception, Equation b, of the "factor of safety" tacitly assumes that only the horizontal thrust of the water is instrumental in moving the center of pressure from e to b, and that the uplift merely lessens the resisting moment.

As the overturning force to be increased is therefore horizontal, and as the length of the line parallel to the overturning resultant and comprehended between the point, b, and the line of action of the resisting force is divided by its segment (comprehended between the actual resultant and the same line of action of the resisting force) to get the ratio, or factor against overturning, it at once follows that the division according to Equation b would be $\frac{u + v}{v}$. Equation a seems preferable, or the "factor of safety" $= \frac{c\ b}{c\ f}$, as in Fig. 7. The "factor of safety," however, is of doubtful value, due to the certain impossibility of the structure's rotation about the point, b; but it may be a useful quantity for comparison at times. The expression (a) may give values less than (b) by as much as one-third, in some cases.

The detailed results of calculations determining the computed sections, A, B, C, and D, of Fig. 6, are given in Table 3; the symbols used therein being fully explained under the subsequent heading, "*Nomenclature.*"

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TABLE 3.—DETAILED RESULTS OF CALCULATIONS DETERMINING THE COMPUTED SECTIONS, A, B, C, AND D.

($c = \frac{2}{3}$; $\Delta = 2\frac{1}{4}$; $6\ T = 2\ 064$.)

Computed cross-section.	Joint No.	H (Head).	H^3	h in feet.	$H + a$ (height).	A_0 in square feet.	A in square feet.	l_0 in feet.	l_0^2	l in feet.	y_0 in feet.	y in feet.	u in feet.	t in feet.	$A_0\ y_0$
A (Read H and a, H_1 and a_1.)	1	8.5	615	18.5	18.5	....	275.5	10.0	100.0	19.8	....	7.7	$\frac{l}{3}$	....	....
	2	19.5	7 440	11.0	29.5	275.5	528.9	19.8	392.5	26.2	7.7	9.6	"	....	2 130
	3	30.5	28 400	11.0	40.5	528.9	849.4	26.2	690.0	32.0	9.6	11.5	"	....	5 070
	4	47.5	107 170	17.0	57.5	849.4	1 475.0	32.0	1 024.0	41.5	11.5	14.5	"	....	9 760
B	1_b	9.1	....	....	11.6	....	69.6	6.0	....	6.0	....	3.0	$\frac{l}{3}$	....	....
	3	33.0	35 937	23.9	35.5	69.6	439.6	6.0	36.0	25.0	3.0	$\frac{l}{3}$	"	1.05	....
	4	50.0	125 000	17.0	52.5	439.6	992.1	25.0	625.0	40.0	$\frac{l}{3}$	$\frac{l}{3}$	"	0.61	....
												$t_3 + t_4$	=	1.66	
C	1_c	16.2	....	....	23.7	....	237.0	10.0	....	10.0	....	5.0	$\frac{l}{3}$	....	....
	3	33.0	35 937	16.8	40.5	237.0	491.0	10.0	100.0	20.3	5.0	⅓	"	1.12	....
	4	50.0	125 000	17.0	57.5	491.0	999.0	20.3	412.0	39.5	⅓	⅓	"	2.75	....
												$t_3 + t_4$	=	3.87	
D	1	11.0	1 331	13.5	13.5	0	75.2	5.0	25.0	6.2	....	2.8	$\frac{l}{3}$	....	...
	3	33.0	35 937	22.0	35.5	75.2	375.0	6.2	38.0	21.1	2.8	$\frac{l}{3}$	"	0.9	....
	4	50.0	125 000	17.0	52.5	375.0	836.0	21.1	446.0	33.1	$\frac{l}{3}$	$\frac{l}{3}$	"	0.4	....
												$t_3 + t_4$	=	1.3	

Mr. Brodie. As brief a statement as possible of the formulas mentioned earlier by the speaker follows. Of these formulas, the following were necessary for determining Sections *A*, *B*, *C*, and *D*:

For Section A, Series D.—Stage I was first used, but a few trial substitutions (necessary because of the form of expression) conclusively showed that only Stage II applied here.

For Section B, Series C.—Stage I applied; but, after using Stage II and calculating *y*, by supplementary formula, it was found necessary to proceed to Stage III for the calculation of Joints 3 and 4.

For Section C, Series C.—Stage I was used for Joint 1_c, and Stage III for Joints 3 and 4.

For Section D, Series A.—Stage I was used, as for Section *A*, and with the same conclusion, except that Stage II applied only to the determination of Joint 1, after which Stage III applied

(It may be stated here that the speaker has used Series *B* satisfactorily, with only the thrust condition, for the rapid determination of bridge abutment dimensions.)

Six series of formulas* will be given and designated by the letters *A*, *B*, *C*, *D*, *E*, and *F*, respectively. Each series conforms to a given set of conditions with respect to external forces.

Series *F* forms a set of general equations for the condition of water overtopping the dam.

Nomenclature.—The following nomenclature will apply to the formulas: "Toe" or "Front" will signify the down-stream face; "Heel" or "Back" will signify the up-stream face. All linear distances are expressed in feet, and all areas in square feet. In connection with the following see Figs. 8 to 13.

L = the width of the top of the dam cross-section;

l = the length of a horizontal joint of masonry, to be determined;

l_0 = the known length of the joint next above the joint of length l;

h = the depth of a course of masonry (the vertical distance between l_0 and l);

P = the line of pressure, reservoir full;

P' = the line of pressure, reservoir empty;

u = the distance from the front edge of the joint, l, to the point of intersection of P with the joint, l, measured parallel to the joint, l;

y = the distance from the back edge of the joint, l, to the point of intersection of P' with the joint, l, measured parallel to the joint, l;

* The demonstration may be found in "High Masonry Dam Design," by C. E. Morrison and O. L. Brodie, Associate Members, Am. Soc. C. E.

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y_0 = the distance from the back edge of the joint, l_0, to the point of intersection of P' with the joint, l_0, measured parallel to the joint, l_0:

r = the distance between P and P' at the joint, l, measured parallel to the joint, l;

γ = the weight of a cubic foot of water, in pounds (62.5 lb.);

γ' = the weight of a cubic foot of mud, in pounds (75 to 90 lb.);

Δ = the ratio of the unit weight of masonry to the unit weight of water;

$\Delta\gamma$ = the weight of a cubic foot of masonry, in pounds;

H = the head of water on the joint, l (vertical distance of the joint, l, below the water surface);

H' = the depth of the earth back-fill over the joint, l, on the front;

H_1 = the head of water on the joint, l, when ice acts at the surface of the water;

$H - H_1$ = the rise of the water level, due to flood, waves, etc., above the normal level for full reservoir;

h_1 = the head of water above the mud level (liquid mud of weight, γ');

h_2 = the head of liquid mud on the joint, l, on the back;

a = the vertical distance from the top of the dam to the surface of the water (flood);

a_1 = the vertical distance from the top of the dam to the surface of the water when ice is considered (a_1 generally exceeds a);

b = the vertical distance from the water surface to the top of the dam when the dam is overtopped;

c = the ratio of upward thrust intensity due to hydrostatic head, H (or H_1, or $h_1 + h_2$) assumed to act at the heel of the joint, l (assumed as $\frac{2}{3}$);

$T\gamma$ = the horizontal ice thrust at the water surface, in pounds;

$D\gamma$ = the horizontal dynamic thrust of the water, in pounds;

$E\gamma$ = the thrust of the earth back-fill, in pounds (on front);

$W_v\gamma$ = the vertical pressure on the inclined up-stream face above the joint, l, in pounds;

A_0 = the total area of the cross-section of the dam above the joint, l_0;

A = the total area of the cross-section of the dam above the joint, l;

t = the batter of the up-stream face for the vertical distance, h;

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$s =$ the distance of the line of action of $W_v \gamma$ from the up-stream edge of the joint, l, measured parallel to the joint, l;

$\delta =$ the angle that $E\gamma$ makes with the horizontal;

$\alpha =$ the angle of slope of the down-stream face of the dam with the horizontal;

$\beta =$ the angle R makes the vertical;

$p =$ the maximum allowable pressure intensity at the toe, in pounds per square foot;

$q =$ the maximum allowable pressure intensity at the heel, in pounds per square foot (p is assumed less than q), p and q may be used to signify the calculated, existent pressure intensities corresponding to P and P', respectively, for the joint, l;

$F = \frac{\gamma H^2}{2} =$ the horizontal static thrust of the water, in pounds;

$W = A \Delta \gamma =$ the total weight of masonry resting on the joint, l, in pounds;

$W_0 = A_0 \Delta \gamma =$ the total weight of masonry resting on the joint, l_0, in pounds;

$R =$ the resultant;

$R' =$ the resultant of the reactions;

$\frac{cHl\gamma}{2} =$ the upward thrust of the water on the base, l.

In Figs. 8 to 12, inclusive, hydrostatic pressures are indicated by triangular and trapezoidal areas included within dotted lines. Ice pressure is indicated to contrast H_1 with H.

As before, a unit length of 1 ft. of dam will be considered. Then the letters, T, D, E, W_v, A, A_0, and H^2 would signify volumes.

It will be observed that, where possible, the several equations will have been cleared of the term $\Delta\gamma$ (thereby simplifying actual calculations).

It will be recalled that the fundamental expression for finding the length, l, of any joint involves M, which must in each case signify the total overturning moment.

The development of a cross-section, by any one of the following series of equations may comprise five stages, each stage representing the introduction of a governing condition. Hence, for each stage, there obtains a main equation for finding the length of the joint, l, each main equation being supplemented by secondary equations for y, u, and t; p and q.

It may be necessary to use more than one of the series of equations in determining a cross-section.

For ready reference, the five stages will be set forth and depicted in order as follows: Mr. Brodie.

Stages in Development.

Stage I.—This stage extends from the top of the dam to the joint where the front face commences to batter. It is the rectangular section. $y > \frac{1}{3} L$; $u \geqq \frac{1}{3} L$ (see Fig. 8). (Ice pressure is purposely omitted in Fig. 8 to prevent a confusion of letters in a small space.)

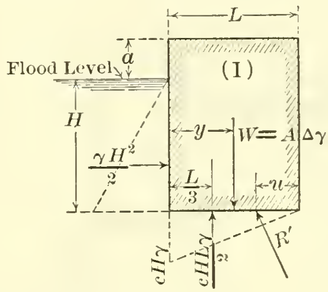

Fig. 8.

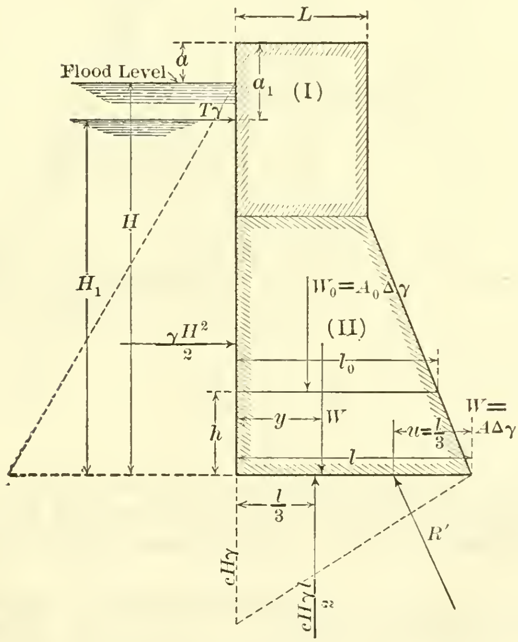

Fig. 9.

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Stage II.—This stage extends from the lower limit of Stage I to the point where the back face commences to batter. $u = \frac{1}{3} l$; $y \geqq \frac{1}{3} l$ (see Fig. 9).

Stage III.—This stage extends from the lower limit of Stage II to the point where the intensity of pressure on the toe has reached the maximum allowable intensity. In this stage, $u = \frac{1}{3} l$; $y = \frac{1}{3} l$ (see Fig. 10).

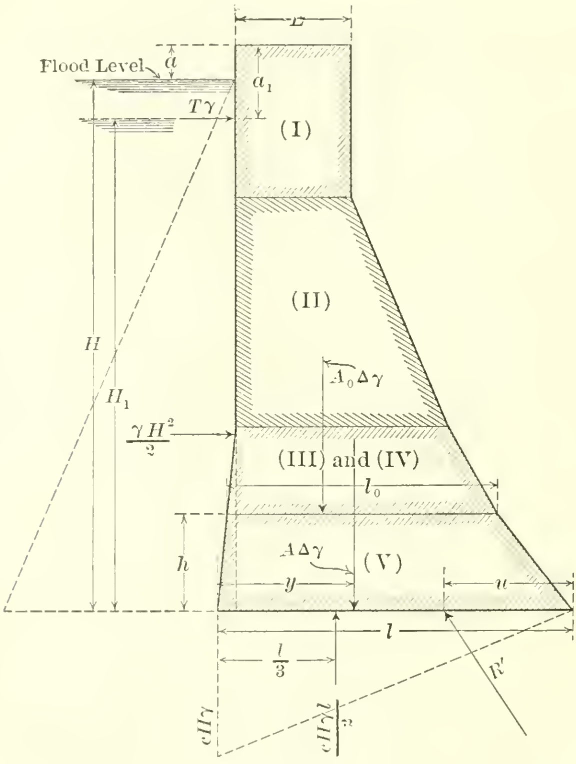

FIG. 10.

Stage IV.—This stage extends from the lower limit of Stage III to the point where the pressure intensity on the heel has reached the maximum allowable intensity. For this stage $u > \frac{1}{3} l$; $y = \frac{1}{3} l$. (See Fig. 10.)

Stage V.—In this stage the limiting intensities of pressure at both toe and heel having been reached, $y > \frac{1}{3} l$; $u > \frac{1}{3} l$. It extends from the lower limit of Stage IV downward. (See Fig. 10.)

The following secondary formulas, supplementary to the main equations of all series, with substitutions as noted, are arranged in order corresponding with the preceding. Mr. Brodie.

Equations Supplementary to All Series.

With condition of hydrostatic upward pressure on base obtaining, substitute the formulas in this column in place of those corresponding, as indicated.

Stage I.

$$u \geqq \tfrac{1}{3} L$$

$$y = \tfrac{1}{2} L$$

$$t = 0$$

$$p = \frac{2 \Delta \gamma A}{L} \left(2 - \frac{3 u}{L}\right)$$

$$p = \gamma \left(\frac{2 \Delta A}{L} - c H\right) \left(2 - \frac{3 u}{L}\right)$$

$$q = \frac{\Delta \gamma A}{L}$$

Stage II.

$$u = \tfrac{1}{3} l$$

$$y = \frac{A_0 y_0 + (l^2 + l\, l_0 + l_0^2) \frac{h}{6}}{A_0 + \left(\frac{l + l_0}{2}\right) h}$$

$$t = 0$$

$$p = \frac{2 \Delta \gamma A}{l}$$

$$p = \gamma \left(\frac{2 \Delta A}{l} - c H\right)$$

$$q = \frac{2 \Delta \gamma A}{l} \left(2 - \frac{3 y}{l}\right)$$

Stage III.

$$u = \tfrac{1}{3} l$$

$$y = \tfrac{1}{3} l$$

$$t = \frac{2 A_0 (l - 3 y_0) - h\, l_0^2}{6 A_0 + h\, (2 l_0 + l)}$$

$$p = \frac{2 \Delta \gamma A}{l}$$

$$p = \gamma \left(\frac{2 \Delta A}{l} - c H\right)$$

$$q = \frac{2 \Delta \gamma A}{l}$$

Stage IV.

$$u = \tfrac{2}{3} l - \frac{p\, l^2}{6 \Delta \gamma A}$$

$$u = \tfrac{2}{3} l - \frac{p\, l^2}{3 \gamma (2 \Delta A - c H l)}$$

$$y = \tfrac{1}{3} l$$

$$t = \frac{2 A_0 (l - 3 y_0) - h\, l_0^2}{6 A_0 + h\, (2 l_0 + l)}$$

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EQUATIONS SUPPLEMENTARY TO ALL SERIES—(*Continued*).

	With condition of hydrostatic upward pressure on base obtaining, substitute the formulas in this column in place of those corresponding, as indicated.
$p = \frac{2\,\Delta\gamma\,A}{l}\left(2 - \frac{3\,u}{l}\right)$ (limiting intensity)	$p = \gamma\left(\frac{2\,\Delta\,A}{l} - cH\right)\left(2 - \frac{3u}{l}\right)$
$q = \frac{2\,\Delta\gamma\,A}{l}$	
Stage V.	
$u = \frac{2}{3}\,l - \frac{p\,l^2}{6\,\Delta\gamma\,A}$	$u = \frac{2}{3}\,l - \frac{p\,l^2}{3\,\gamma\,(2\,\Delta\,A - c\,H\,l)}$
$y = \frac{2}{3}\,l - \frac{q\,l^2}{6\,\Delta\gamma\,A}$	

$$t = \frac{A_0\,(4\,l - 6\,y_0) + \left(h - \frac{q}{\Delta\gamma}\right) l^2 + (l - l_0)\,h\,l_0}{6\,A_0 + h\,(2\,l_0 + l)}$$

$p = \frac{2\,\Delta\gamma\,A}{l}\left(2 - \frac{3\,u}{l}\right)$ (limiting intensity)	$p = \gamma\left(\frac{2\,\Delta\,A}{l} - cH\right)\left(2 - \frac{3\,u}{l}\right)$
$q = \frac{2\,\Delta\gamma\,A}{l}\left(2 - \frac{3\,y}{l}\right)$ (limiting intensity)	

If T enters the following formulas, H (above) becomes H_1 (see Figs. 9 and 10).

SERIES A.

Overturning moment due to horizontal static water pressure on back of dam only.

Stage I.

$$H = \sqrt[3]{\Delta\,L^2\,(H + a)}.$$

Stage II.

$$l^2 + \left(\frac{4\,A_0}{h} + l_0\right) l = \frac{1}{h}\left(\frac{H^3}{\Delta} + 6\,A_0\,y_0\right) + l_0^2.$$

Stage III.

$$l^2 + \left(\frac{2\,A_0}{h} + l_0\right) l = \frac{H^3}{\Delta\,h}.$$

Stage IV.

$$l^2 = \frac{\gamma\,H^3}{p}.$$

Stage V.

$$\left(\frac{p + q}{h\,\Delta\gamma} - 1\right) l^2 - \left(\frac{2\,A_0}{h} + l_0\right) l = \frac{H^3}{\Delta\,h}.$$

SERIES B.

Overturning moment due to:

(*a*) Horizontal static water pressure on back, and

(*b*) Ice pressure applied at distance, a_1, from top.

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Stage I.

$$H_1 = \sqrt[3]{\Delta (H_1 + a_1) L^2 - 6 T H_1}.$$

Stage II.

$$l^2 + \left(\frac{4 A_0}{h} + l_0\right) l = \frac{1}{h} \left(\frac{H_1^3 + 6 T H_1}{\Delta} + 6 A_0 y_0\right) + l_0^2.$$

Stage III.

$$l^2 + \left(\frac{2 A_0}{h} + l_0\right) l = \frac{1}{\Delta h} \left(H_1^3 + 6 T H_1\right).$$

Stage IV.

$$l^2 = (H_1^3 + 6 T H_1) \frac{\gamma}{p}.$$

Stage V.

$$\left(\frac{p + q}{h \Delta \gamma} - 1\right) l^2 - \left(\frac{2 A_0}{h} + l_0\right) l = \frac{1}{\Delta h} (H_1^3 + 6 T H_1).$$

SERIES C.

Overturning moment due to:

(a) Horizontal static water pressure on back.

(b) Upward water pressure on base. Pressure intensity decreasing uniformly from $c H \gamma$ at heel to zero intensity at toe.

Stage I.

$$H = \sqrt[3]{L^2 [\Delta (H + a) - c H]}.$$

Stage II.

$$\left(1 - \frac{c H}{\Delta h}\right) l^2 + \left(\frac{4 A_0}{h} + l_0\right) l = \frac{1}{h} \left(\frac{H^3}{\Delta} + 6 A_0 y_0\right) + l_0^2.$$

Stage III.

$$\left(1 - \frac{c H}{\Delta h}\right) l^2 + \left(\frac{2 A_0}{h} + l_0\right) l = \frac{H^3}{\Delta h}.$$

Stage IV.

$$l^2 = \frac{\gamma H^3}{p}.$$

Stage V.

$$\left(\frac{p + q}{h \Delta \gamma} - 1\right) l^2 - \left(\frac{2 A_0}{h} + l_0\right) l = \frac{H^3}{\Delta h}.$$

SERIES D.

Overturning moment due to:

(a) Horizontal static water pressure on back (head $= H_1$).

(b) Ice pressure applied at distance, a_1, from top.

(c) Upward water pressure on base. Pressure decreasing uniformly from $c H_1 \gamma$, at heel to zero intensity at toe.

Stage I.

$$H_1 = \sqrt[3]{L^2 [(H_1 + a_1) \Delta - c H_1] - 6 T H_1}.$$

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Stage II.

$$\left(1 - \frac{c\,H_1}{\Delta\,h}\right) l^2 + \left(\frac{4\,A_0}{h} + l_0\right) l$$
$$= \frac{1}{h}\left(\frac{H_1^3 + 6\,T\,H_1}{\Delta} + 6\,A_0 y_0\right) + l_0^2.$$

Stage III.

$$\left(1 - \frac{c\,H_1}{\Delta\,h}\right) l^2 + \left(\frac{2\,A_0}{h} + l_0\right) l = \frac{1}{\Delta\,h}(H_1^3 + 6\,T H_1).$$

Stage IV.

$$l^2 = \frac{\gamma\,(H_1^3 + 6\,T H_1)}{p}.$$

Stage V.

$$\left(\frac{p+q}{h\,\Delta\gamma} - 1\right) l^2 - \left(\frac{2\,A_0}{h} + l_0\right) l = \frac{H_1^3 + 6\,T H_1}{\Delta h}.$$

SERIES E.

Ice pressure neglected in Fig. 11. Overturning moment due to:

(*a*) Horizontal static water pressure on back (head $= h_1$).

(*b*) Ice pressure applied at distance, a_1, from top.

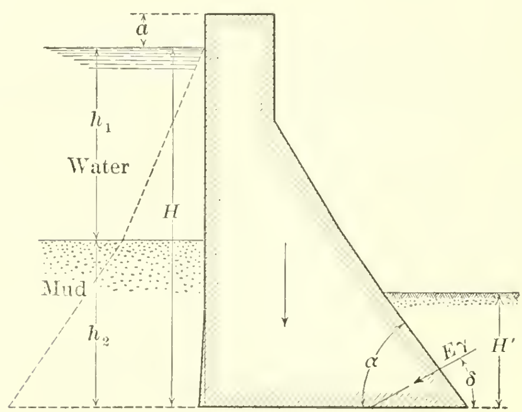

FIG. 11.

(*c*) Upward water pressure on base; pressure intensity decreasing uniformly from $c(h_1 + h_2)\gamma$ at heel to zero intensity at toe.

(*d*) Mud (liquid) pressure on back (head h_2), commencing at distance, h_2, above joint in question. Weight of mud $= \gamma'$. As before, if T be equated to zero, a_1 becomes equal to a in formulas.

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Stage I.—(h_1 of known value, h_2 to be determined).

$$\frac{h_2^3 \gamma'}{\gamma} + (h_1 + h_2)\,[3\,h_1 h_2 + 6\,T + L^2\,(c - \varDelta)] = L^2 a_1 \varDelta - h_1^3.$$

Stage II.

$$\left[1 - \frac{c\,(h_1 + h_2)}{\varDelta\,h}\right] l^2 + \left(\frac{4\,A_0}{h} + l_0\right) l = \frac{1}{\varDelta\,h}\left[(h_1 + h_2)(3\,h_1 h_2 + 6\,T) + h_1^3 + \frac{h_2^3 \gamma'}{\gamma} + 6\,A_0 y_0 \varDelta\right] + l_0^2.$$

For trapezoidal section at top, make $A_0 = 0$, and $y_0 = 0$, and $l_0 = L$ in Stage II. This applies generally.

Stage III.

$$\left[1 - \frac{c\,(h_1 + h_2)}{\varDelta\,h}\right] l^2 + \left(\frac{2\,A_0}{h} + l_0\right) l = \frac{1}{\varDelta\,h}\left[(h_1 + h_2)(3\,h_1\,h_2 + 6\,T) + h_1^3 + \frac{h_2^3 \gamma'}{\gamma}\right].$$

Stage IV.

$$l^2 = \frac{\gamma}{p}\left[(h_1 + h_2)(3\,h_1\,h_2 + 6\,T) + h_1^3 + \frac{h_2^3 \gamma'}{\gamma}\right].$$

Stage V.

$$\left(\frac{p + q}{h\,\varDelta\gamma} - 1\right) l^2 - \left(\frac{2\,A_0}{h} + l_0\right) l = \frac{1}{\varDelta\,h}\left[(h_1 + h_2)(3\,h_1\,h_2 + 6\,T) + h_1^3 + \frac{h_2^3 \gamma'}{\gamma}\right].$$

Series F.

This series consists of general formulas for a number of imposed conditions of loading. For any given case, the terms or factors expressing those conditions not appertaining must be eliminated by equating them to zero. (See Fig. 12.)

Conditions for General Formulas.—Overturning moment due to:

(a) Horizontal static water pressure on back (head $= h_1$).

(b) Upward water pressure on base; pressure intensity decreasing uniformly from $c\,H\gamma$, or $c\,(h_1 + h_2)\gamma$, at heel to zero intensity at toe.

(c) Mud (liquid) pressure on back (head h_2) as before.

(d) Dynamic pressure of water, $D\gamma$.

(e) Water flowing over top of dam, weight of water, of depth b, on top of dam being neglected.

*For condition of water not overtopping dam, $b = 0$ and $D = 0$.

* For water surface below top of dam, Series E, containing a, must be used.

Mr. Brodie.

For condition of no dynamic pressure, $D = 0$.

For condition of no upward water pressure, $c = 0$.

For condition of no mud (that is, mud being replaced by water) make $h_2 = 0$, $h_1 = H$.

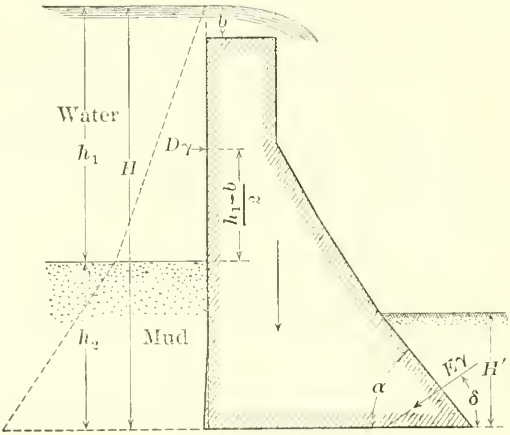

FIG. 12.

Stage I.—Rectangular cross-section at top, or rectangular dam, $l = l_0 = L$. This may fall under either of two cases, namely:

Case (1)

Condition: $h_1 = H$; $h_2 = 0$.

$$H^3 + H\left[+3\,D - 3\,b^2 + L^2\,(c - \Delta\,)\right] = b\,(3\,D - 2\,b^2 - L^2\,\Delta).$$

Case (2)

Condition: h_1 of known value; h_2 to be determined.

$$\frac{h_2^3\,\gamma'}{\gamma} + 3\,D\,h_2 + (h_1 + h_2)\left[3\,h_1\,h_2 + 3\,D - 3\,b^2 + L^2\,(c - \Delta)\right]$$
$$= b\,(3\,D - 2\,b^2 - L^2\,\Delta) - h_1^3.$$

As in the preceding series, the value of H or h_2, of Stage I, may be determined by several successive trial substitutions.

Stage II.—(*a*) Trapezoidal cross-section at top of dam or trapezoidal dam (spillway) front face battered. ($A_0 = 0$, $l_0 = L$, and $y_0 = 0$.) Note: For a triangular dam, $l_0 = 0$, also.

$$\left[1 - \frac{c\,(h_1 + h_2)}{\Delta\,h}\right] l^2 + L\,l = \frac{1}{\Delta\,h}\left[(h_1 + h_2)\;(3\,h_1\,h_2 - 3\,b^2)\right.$$
$$\left. + \, h_1^3 + \frac{h_2^3\,\gamma'}{\gamma} + 2\,b^3 + 3\,D\,(h_1 + 2\,h_2 - b)\right] + L^2.$$

(*b*) Trapezoidal section continued (front face battered). Mr. Brodie.

$$\left[1 - \frac{c\,(h_1 + h_2)}{\Delta\, h}\right] l^2 + \left(\frac{4\, A_0}{h} + l_0\right) l = \frac{1}{\Delta\, h}\left[(h_1 + h_2)\right.$$
$$(3\, h_1\, h_2 - 3\, b^2) + h_1^3 + \frac{h_2^3\, \gamma'}{\gamma} + 2\, b^3 + 3\, D\, (h_1 + 2\, h_2 - b)$$
$$\left. + 6\, A_0\, y_0\, \Delta\right] + l_0^2.$$

Stage III.—Both faces battered.

$$\left[1 - \frac{c\,(h_1 + h_2)}{\Delta\, h}\right] l^2 + \left(\frac{2\, A_0}{h} + l_0\right) l = \frac{1}{\Delta\, h}\left[(h_1 + h_2)\right.$$
$$\left.(3\, h_1\, h_2 - 3\, b^2) + h_1^3 + \frac{h_2^3\, \gamma'}{\gamma} + 2\, b^3 + 3\, D\, (h_1 + 2\, h_2 - b)\right].$$

Stage IV.—Limiting intensity of pressure, p, introduced.

$$l^2 = \frac{\gamma}{p}\left[(h_1 + h_2)\,(3\, h_1\, h_2 - 3\, b^2) + h_1^3\right.$$
$$\left. + \frac{h_2^3\, \gamma'}{\gamma} + 2\, b^3 + 3\, D\, (h_1 + 2\, h_2 - b)\right].$$

Stage V.—Limiting intensities, p and q.

$$\left(\frac{p + q}{h\ \ \Delta\gamma} - 1\right) l^2 - \left(\frac{2\, A_0}{h} + l_0\right) l = \frac{1}{\Delta\, h}\left[(h_1 + h_2)\,(3\, h_1\, h_2 - 3\, b^2)\right.$$
$$\left. + h_1^3 + \frac{h_2^3\, \gamma'}{\gamma} + 2\, b^3 + 3\, D\, (h_1 + 2\, h_2 - b)\right].$$

The increased number of overturning loads, then, tend to render the right-hand members of the various equations more involved; though, after a little practice, one may easily carry through a design with surprising rapidity. The slide rule may be used to great advantage, and it is suggested that the results be tabulated as determined, in some such form as the values for Sections *A, B, C,* and *D,* in Table 3.

The effect on the calculation of a cross-section of back-fill on the down-stream face, of course, could be cared for by introducing that condition into the preceding series of equations; but as this effect, as computed, would be, in any case, largely dependent on assumptions which may vary widely, and, as the placing of back-fill is generally a later consideration with respect to construction, the propriety of such introduction at that stage of design is questionable.

In the following formulas for investigation, therefore, the general conditions of an earth thrust acting at the down-stream face, and of a vertical component of thrust of material on the up-stream, inclined face of the dam are introduced.

By any of these formulas, the position of the line of resistance for any given cross-section and respective conditions may be determined with regard to any horizontal joint and its down-stream edge, the value of u being the quantity sought.

Mr. Brodie.

Any condition may be disregarded by equating its term to zero.

The first expression below contains all the conditions heretofore considered, with the additional ones just stated; and from it follow the succeeding expressions for u. It should be remembered that the term, T, cannot be co-existent in any expression for stability with b, and therefore with D. Nevertheless, all these terms are written with the understanding that the proper eliminations be always made. Three general group equations will be written.

Formulas for Investigation.

First, Conditions of retained mud, water, overtopping, etc.—(See Fig. 12.)

$$u = l - y - \frac{\left\{ \begin{array}{l} (h_1 + h_2)\,[3\,h_1 h_2 + 6\,T - 3\,b^2 + c\,l\,(3\,y - l)] + h_1^3 \\ \quad + \dfrac{h_2^3\,\gamma'}{\gamma} + 2\,b^3 + 3\,D\,(h_1 + 2\,h_2 - b) - 6\,W_v\,(y - s) \\ \qquad + 6\,E\left[(l - y)\,\text{sin.}\,\delta - \dfrac{H'}{3}\,\dfrac{\text{sin.}\,(\delta + a)}{\text{sin.}\,a}\right] \end{array} \right\}}{6\,(W_v + E\,\text{sin.}\,\delta + A\,\Delta) - 3\,c\,(h_1 + h_2)\,l}.$$

Whence, for conditions of retained mud, water, etc., but no overtopping, by making $b = 0$ and $D = 0$, there follows (see Fig. 11),

$$u = l - y - \frac{\left\{ \begin{array}{l} (h_1 + h_2)\,[3\,h_1\,h_2 + 6\,T + c\,l\,(3\,y - l)] + h_1^3 + h_2^3\,\dfrac{\gamma'}{\gamma} \\ \quad - 6\,W_v\,(y - s) + 6\,E\left[(l - y)\,\text{sin.}\,\delta - \dfrac{H'}{3}\,\dfrac{\text{sin.}\,(\delta + a)}{\text{sin.}\,a}\right] \end{array} \right\}}{6\,(W_v + E\,\text{sin.}\,\delta + A\,\Delta) - 3\,c\,(h_1 + h_2)\,l}.$$

From this last expression for u, for conditions of retained water, etc., but neither mud nor overtopping, by making $h_1 = H_1$, $h_2 = 0$, there is obtained:

$$u = l - y - \frac{\left\{ \begin{array}{l} 6\,T + H_1^2 + c\,l\,(3\,y - l) - \dfrac{6\,W_v}{H_1}\,(y - s) \\ \quad + \dfrac{6\,E}{H_1}\left[(l - y)\,\text{sin.}\,\delta - \dfrac{H'}{3}\,\dfrac{\text{sin.}\,(\delta + a)}{\text{sin.}\,a}\right] \end{array} \right\}}{6\left(\dfrac{W_v + E\,\text{sin.}\,\delta + A\,\Delta}{H_1}\right) - 3\,c\,l}.$$

As in equations for design, when $T = 0$, $H_1 = H$. If H' is of such depth that the down-stream batter of the cross-section varies considerably, an approximate solution is possible by assuming some average batter for the lower portion. The expression for the earth thrust is general, as is seen. After u is determined for each joint, the intensities of maxima pressures can be determined for the given cross-

Mr. Brodie.

section, the general expression for p, corresponding to the above expressions for u, being:

$$p = \frac{2\,\gamma}{l}\left[W_v + E \sin.\,\delta + A\,\Delta - \frac{c\,(h_1 + h_2)}{2}\,l\right]\left(2 - \frac{3\,u}{l}\right).$$

In connection with the computation for the value of y in an investigation, as indicated above, it is necessary to obtain the position of the centroid of a trapezoid with respect to the back, or up-stream edge of the joint in question. The following expression for x, in connection with Fig. 13, may prove convenient:

$$x = \frac{(l^2 + l\,l_0 + l_0^2) + t\,(l + 2\,l_0)}{3\,(l + l_0)}.$$

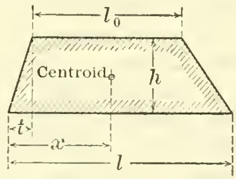

Fig. 13.

If it is desirable to consider tension as active in the joint, and if p_t' is the intensity, in tons per square foot, at the down-stream end of the joint, and p_t'' is the intensity at the up-stream edge of the joint, p, above, which, as written, is in pounds per square foot, will take the form:

$$p_t' = \frac{1}{16\,l}\left[W_v + E \sin.\,\delta + A\,\Delta - \frac{c\,(h_1 + h_2)\,l}{2}\right]\left[2 - \frac{3\,u}{l}\right]$$

and

$$p_t'' = \frac{1}{16\,l}\left[W_v + E \sin.\,\delta + A\,\Delta - \frac{c\,(h_1 + h_2)\,l}{2}\right]\left[\frac{3\,u}{l} - 1\right].$$

In the case where there is liquid mud only on the back, W_v becomes equal to $\frac{A'\,\gamma'}{\gamma}$, where A' is the area of the superimposed mud.

Mr. Dunham.

H. F. Dunham, M. Am. Soc. C. E. (by letter).—Since the failure of the dam at Austin, Pa., the author and a large number of contributors to technical journals appear to agree on two important considerations. The author, in tersest English, voices his own and the ideas of all those writers, as follows:

"The effect of this upward pressure, however, must be counteracted, either by increasing the section of the dam or by increasing its height above the water level in the reservoir, or by both."

It is the writer's purpose to hint at a third method of counteracting upward pressure. Its value can be compared with the two first named. There can be no question that a dam should have a good foundation. Firm and impervious rock should always be safe and satisfactory. Strong, tough, and pervious rock should also be safe and fairly satisfactory, provided it affords a good anchorage. Thin layers of stone, and silt or mud, are excluded from this definition.

It often happens that water under pressure passes through a block of sandstone in quantity sufficient to make, not a brook, but a small stream. No one is disturbed, however, by any fear of cleavage, Mr. Dunham. splitting, or other fracture of the block, because he knows that the cohesive strength of the stone is greatly in excess of the disrupting force of the water. As a matter of fact, the resistance of the block to overturning increases with the presence of the water in the porous stone, for the weight is increased. Imagine such a block to be extended in dimensions until it is large enough for an Austin Dam or a New Croton Dam. Would it be in the least degree unsafe, provided the tensile strength of the stone was still in excess (by a proper safety factor) of the bursting force of the water, and provided, of course, that it was held to the foundation rock by equally cohesive anchorage?

In Boston the granite columns from the old Court House are being removed, and it is necessary to take them away in pieces; but those columns are not found to be in pieces, although they have been exposed to a wide range of temperature for nearly a hundred years. Imagine the entire Court House as a block of granite. Would it have been split up by the frost or by changes in temperature? Scattered through the State of Maine are still larger blocks of granite, free from weather defects, although they have been exposed for intervals of time reckoned as ages in geology. It may be urged, however, that horizontal cleavage due to change of temperature in large work cannot be avoided. At what measured interval must such cleavage planes occur? Examine vertical walls in quarries, and natural escarpments of granite and sandstone, or other firm formations, and note the distance between the horizontal cleavage planes which Nature was obliged to adopt for her standard. How insignificant are all dams compared with the masses of sandstone in Western cañons, hundreds of feet in thickness and without a defect.

Is there any real trouble about placing in an Austin Dam, concrete of a quality to resist the uplift pressure, say, 20 or 30 lb. per sq. in.; or in many larger dams? Is there any real trouble about joining the work of one day to the work of the previous day or week, so that it shall be as strong as any other part? Such a dam, with the tensile strength of the material in excess of the uplift stresses by a proper safety factor, and designed without correction for uplift, would have no element of danger, if securely anchored, though the material were as porous as are many grades of sandstone.

If necessary, in construction, to avoid exposing the outer walls of a dam to low temperatures while the structure elsewhere is subjected to the high temperature due to the setting of the cement, a method less expensive than thickening the walls could be provided, and nearly uniform temperatures could be maintained as the work progressed.

Concrete and cyclopean masonry have been improved, and may be further improved, until they approach in quality a second or third-class sandstone, and later some of the firmer rocks. Mr. Dunham.

The writer is aware that some authorities place the tensile strength of rather lean (1: 2: 5) concrete at 350 lb. per sq. in., and of sandstone as high as 500 lb. per sq. in., and other authorities recommend a zero factor for tensile strength in all masonry.

The sense of it all is this: Good construction is not subject to the defects which are common and often destructive enough in poor work. Everywhere the tendency should be to bring the quality up to the requirements of a good design, and not to fit a design to work of poor quality.

C. Elmore Smith, Assoc. M. Am. Soc. C. E. (by letter).—The present-day development of large storage reservoirs, for domestic water supply, irrigation and power purposes, flood prevention, etc., has compelled the engineer to consider more and more carefully the masonry dam, and to evolve formulas which will take into account more and more fully the due importance of the supposed distribution of stresses in such structures; but, are we not still groping somewhat in the dark? It would seem to be high time that some large engineering body, educational institution, or branch of the National Government make an exhaustive study of the subject by testing to failure large models built under practical working conditions. This might well be done, and would at least get rid of all the more or less empirical formulas which are largely the result of development, along purely mathematical and theoretical lines, since the first dam was built. Mr. Smith.

In regard to hydrostatic pressure under the base of a dam, as Mr. Harrison says, this can only occur when the foundation rock is of a porous or stratified nature, "with well-defined horizontal seams;" and he has covered the ground in saying:

"Generally, it will be found cheaper to make large expenditures to provide a cut-off in the foundation * * * located at the heel of the dam."

When one considers the cost of the entire enterprise and the disastrous results of a failure, including loss of life and property, and the interrupted use of the reservoir, for whatever purpose constructed, too great a stress cannot be laid on the importance of first carrying the whole foundation to rock capable of supporting the great weight of the dam, and, further, of carrying the cut-off wall even to extreme depths to ensure the interception of all possible water-bearing strata or fissures. The question of the masonry being pervious to water, and thus creating hydrostatic pressure within the body of the dam itself, should not be discussed for a moment, as a masonry dam, when properly constructed, can and should be practically impervious.

In the design of masonry dams, the element of stress due to ice

Mr. Smith. pressure has interested engineers, and has been considered by them in different ways in making the plans for many of the structures now in existence.

It might be well to ask: are there any records of failures of dams due to ice pressure? Edward Wegmann, M. Am. Soc. C. E., mentions only one (that of the Minneapolis Mill Company, in 1899), and says that "there were special conditions in this instance which remove it from the ordinary case of ice pressure against a dam."

The Board of Experts, appointed to study the profile proposed by Mr. B. S. Church and the late Alphonse Fteley, Past-President, Am. Soc. C. E., for the Quaker Bridge Dam, adopted 43 000 lb. per lin. ft. as the extreme ice pressure. This and what they termed the effect of a possible transition wave caused them to add a thickness of about 9 ft. to the top section of the profile as proposed. The New Croton Dam—the Quaker Bridge project as finally located—was built, however, on substantially the first profile, without this added thickness. The profile of the extension of this dam was made 2.1 ft. thicker at an elevation of 100 ft. above the river bed, decreasing to zero at elevations about 40 ft. above and below this level, because J. Waldo Smith, M. Am. Soc. C. E., then Chief Engineer, believed the profile at this level to be deficient in strength. Did Mr. Smith consider ice pressure, in applying this added thickness?

In the design of the Cross River Dam, the ice pressure was taken at 24 000 lb. per lin. ft., and for the Croton Falls Dam, immediately thereafter, 30 000 lb. per lin. ft. was used. In the latter case, Alfred D. Flinn, M. Am. Soc. C. E., in charge of the design, explains this added 6 000 lb. by stating that "the configuration of the reservoir makes ice thrusts more probable."

These few cases in which ice pressure was considered have come under the writer's observation by his direct connection with the work.

If it is assumed that the probable extreme expansive pressure of ice is its crushing strength less a compression of from 6 to 30%, it would seem that there is need of reliable data and experiments in regard to this element of design, as the best data the writer can find (tests by the U. S. Engineer Corps in 1880) give the crushing strength of ice as varying from 100 to 1 000 lb. per sq. in., a difference too wide to permit of determining any very definite coefficient.

The design of dams has been an ever-recurring problem to the engineer, since the earliest times. From the earthen dams of ancient history to the first masonry structure, probably in Spain, about the Sixteenth Century, through the clumsy, faulty, and extravagant designs of the first Spanish dams and those of France early in the Nineteenth Century, which seem to have been built with very little if any conception of a theory of design, to the modern dams of the French, English, and Americans, the subject has been studied.

Mr. Smith.

Through the researches of unknown early hydraulic engineers and of De Sazilly in 1853, Delocre, Rankine, Harlocher, Crugnola in 1882, and a host of others, the search for a rational theory of the design of masonry dams has been prosecuted; yet, when the plans for the great Quaker Bridge Dam were prepared under the direction of the late Isaac Newton, M. Am. Soc. C. E., assisted by the late E. S. Chesbrough, J. W. Adams, and J. B. Francis, Past-Presidents, Am. Soc. C. E., and the Aqueduct Commission ordered Mr. B. S. Church, its Chief Engineer, to make new and complete research on the problem of the design of a high masonry dam (Mr. Church assumed this important undertaking with the assistance of the late Alphonse Fteley, Past-President, Am. Soc. C. E., and J. P. Davis, M. Am. Soc. C. E.), very few new data were obtainable.

After protracted investigations, and much mathematical study by Mr. Wegmann, assisted by the late Ira A. Shaler, M. Am. Soc. C. E., a profile was finally evolved and presented to the Aqueduct Commissioners, accompanied by an exhaustive report, showing the result of all these investigations and the reasons for the profile proposed.*

Thus it is seen that, as late as 1892, engineers were still at odds over the question of the proper design of a masonry dam.

Then came the investigations of the Metropolitan Water Board of Boston, the studies for the proposed large dams for the irrigation projects of the United States Reclamation Service, those for the dams proposed on the Catskill Aqueduct System, and those for the dams in connection with the Panama Canal.

Are engineers now any nearer the ultimate solution of the rational theory of the design of structures of this class? Certain theoretical and mathematical deductions have been advanced; models of dams have been built, of wood, rubber, stone, and various other materials, all on a small scale, and these models have been tested in different ways; but these investigations assume conditions which are largely theoretical, namely, perfect elasticity of materials, uniform temperatures, uniform quality of materials and workmanship, and many ideals which are never obtained in practice.

Finally (though this matter is not strictly pertinent to the discussion of this paper), the writer wishes to urge upon the Profession his belief that, as masonry dams are generally monumental structures, they should be built as such. Only the best and most permanent architectural effects should be countenanced. No cheap "near stone" facing or mere veneering should be attempted or permitted, especially when dealing with a material like concrete, the durability of the face finish of which, to stand for centuries the rigors of this climate, may be questioned.

* Further historical matter submitted by Mr. Smith is omitted because the subject is covered fully in Mr. Wegmann's discussion.

Mr. Smith. Have not the engineers of Europe gone far ahead of those of America in this consideration; and is it not time for American engineers to add the element of beauty in architectural effects to their strictly utilitarian designs?

Mr. Meem. J. C. Meem, M. Am. Soc. C. E. (by letter).—The writer has not followed the discussion of this paper closely, but has been interested in the paper itself, and in some of the discussion, particularly that by Mr. Godfrey, in which he makes a strong plea for the design of dams to include provision against uplift at full pressure over the whole area, and quotes in confirmation of his position, the following:

"Mr. J. B. Francis held that solid concrete deposited on bed rock would be lifted or floated, and to prove this, placed a pipe provided with a gauge, in the concrete of the dam and found that the gauge registered the full pressure."

The writer has for some time tried to have engineers distinguish between full pressure over the whole area, and full pressure on a percentage of the whole area. A dam cannot rest on rock (or ground either, for that matter) and be separated from it by even a minute layer or film of water over its whole area. That is, there must be areas of absolute contact distinguished from the porous areas through which on one side there is a flow of water causing pressure against a percentage of the area on the other.

The quantity of water which seeps into or sweats through a dam gives in some degree the relative porosity of the dam; and, assuming that another percentage of pores, probably small, does not find an outlet, it may be assumed that this latter percentage represents the area over which full pressure is exerted as uplift in the dam itself, and that this may vary from 1 to 10% of the whole, according to the material of which the dam is built. In the same way, considering the material on which it is built, the water through a percentage of the seams (or voids) does not find an outlet, and here also it may be assumed that full pressure is exerted. The percentage in this case may be assumed to vary (roughly) from 1 to 40% of the whole area. The sum of these percentages can never by any possibility be more than 50, and in practice may never reach more than 5 or 10. For instance, if a table with a polished surface is placed in a receptacle and on it there is placed a block with a polished surface in contact with that of the table, then, assuming that the block has a specific gravity slightly less than that of water, it will not float when the receptacle is filled with water, because there is no area to make it buoyant. If, then, holes, the aggregate area of which is a little more than sufficient to cause the block to be buoyant, be drilled through the table, the block will float. If again, holes be drilled through the block to correspond exactly with those in the table, again the block will not float; although, if one or two of these holes be plugged with a gauge they will show full hydrostatic pressure. Exactly the same condition should obtain in a dam on rock or firm ground, except that the contact of the two polished surfaces is replaced by the contact between concrete and stone, or concrete and the soil, which is very intimate. Mr. Meem.

Every existing chamber which has no outlet (or any gauge placed at the foundation or in the material of the dam) will undoubtedly in time show full hydrostatic pressure. That is, it will be in effect the chamber of a hydraulic jack.

When concrete which has been deposited on rock bursts up (if it ever does) it will not do so until the full pressure, multiplied by what may be defined as the porous area, is greater than the weight of the concrete. The writer's attention has been called to a case in which a pump-house has a floor of concrete resting on ground in which the weight of the floor is not heavy enough to resist full pressure over the whole area, and yet the floor is there. He believes that many cases of floors bursting up will be found to be caused by the pressure being sufficiently in excess of the weight to offset the diminished area against which the pressure initially acts; still, any of these cases may not in themselves be conclusive as applied to masonry dams on rock. It may be of interest to note that the gauges in the Battery Tunnel always showed full hydrostatic pressure, and yet the tunnel, with a relative buoyancy in excess of its weight of more than 3 tons per ft., sank under any disturbance of the surrounding material during construction.

Finally, the writer wishes to emphasize the fact that he does not desire to take issue with those who would use in design large factors of safety against all possible contingencies, but does wish to make clear the fact that engineers should use their technical knowledge and experience to design as nearly correctly as possible, and then should provide against contingencies by using such factors of safety as judgment or experience may dictate.

W. J. Douglas, M. Am. Soc. C. E. (by letter).—*Uplift.*—It seems rational to disregard uplift if the dam is built on impervious rock, but, even in such case, a cut-off wall, having a depth and width of approximately one-tenth the height of the dam, should be let into the rock at the heel. Further, the line of pressure, under maximum water-load conditions, should lie well within the middle-third. Mr. Douglas.

The experiments made by Ottley and Brightmore* indicate the probability that there is a greater tendency toward tension at the heel of a masonry dam than is indicated by the common straight-line distribution of pressure, which ignores the elasticity of the material. If, therefore, a dam is designed so that the line of pressure is

* *Minutes of Proceedings*, Inst. C. E., 1908.

Mr. Douglas. practically at the limit of the middle-third, there may be a tendency for tension at the heel which the masonry is incapable of transmitting to the foundation. In this case, the bed joint would open, and actual uplift would take place. Based on the experiments referred to, the writer would suggest a slight batter at the back of a dam, say 1 to 12, extending from the lower third point to the base. This masonry would be added to the theoretically determined section, the line of pressure in which was at or near the limit of the middle-third.

In regard to the question of the probable imperviousness of rock, the designer is often forced to assume, on the basis of borings or test pits, that the entire dam site, when stripped, will be in accordance with the small area explored, but this is frequently incorrect. If the designer assumes no uplift, it is incumbent on him to verify the safety of such an assumption by observation of the uncovered foundation bed.

The suggested back batter, in conjunction with a reasonably deep cut-off wall, is all that is necessary to guard against uplift under a dam resting on an impervious foundation, provided the foundation is properly cleaned and the masonry well bonded to it. It should be borne in mind, however, that in all gravity dam design engineers are working with a comparatively low factor of safety, and this is justifiable only when the conditions are known with a high degree of precision. In the case of a gravity dam, even on an impervious bottom, the designer is forced to assume a unit weight of the material and a distribution of the stresses, neither of which is quite correct, and, for this reason, if the suggested back batter is omitted, the line of pressure under maximum water-load conditions should lie well within the middle-third.

If the foundation bed is pervious, every effort, of course, should be made to stop seepage, or flow of water, by the construction of a cut-off wall carried down to impervious material; and adequate drains should be provided below (down stream from) the heel, so that any water passing the cut-off wall may be carried off without causing material uplift. The intensity of this uplift depends entirely on the resistance which the water meets after it enters below the dam and before its free discharge there. If the water meets with no resistance, there is no uplift; whereas, if there were no drains, and if a second cut-off wall were provided at the down-stream toe, and this latter wall were carried to an impervious bed, the entire base of the dam might be subjected to an uplift equal to the full hydrostatic head. It is desirable, therefore, not to have a toe cut-off wall unless it is imperative, as in the case of a spillway section; and, when this is so, this wall should not be carried down to impervious strata, unless in the design of the dam full uplift has been considered over its entire base.

If the cut-off wall at the heel cannot be carried down to impervious strata, the designer would best assume full uplift at the heel and zero

Mr. Douglas.

at the toe. This is only necessary when the foundation bed is generally pervious, as occasional springs can be cared for by drainage or possibly by grouting. In reference to this matter, however, it might be well to keep in mind the fact that precautions which may be ample to care for springs under practically no head, prior to the dam construction, may be entirely inadequate under a high head.

In regard to the construction of a gravity dam on a bed which is so porous that it might transfer full hydrostatic head for the full area of the base, and where cut-off walls cannot be constructed to sufficient depth to decrease this head materially, it seems that such a site is unsafe for a gravity section, whereas the buttress and the hollow concrete dam offer types particularly advantageous under such conditions. The buttress type is desirable when the dam is to rest on pervious rock of sufficient strength to withstand the concentrated pressures brought by the buttresses. The hollow reinforced concrete dam would best be used where the ground is pervious and too soft to carry the heavy pressures which would be brought on it by the gravity dam and more particularly by a buttressed one.

In regard to uplift within the masonry itself, there is no doubt that, in even the most compact and impervious masonry, there is a certain amount of it, but as thin reinforced concrete slabs of hollow dams are withstanding high heads without even moisture on the under side, it seems highly improbable that, in a well-built gravity dam, uplift need be taken into consideration. In a badly constructed dam, or in one in which improper materials are used, full uplift might occur at the heel of any section with zero uplift pressure at the toe. To assume full uplift at the toe would be to assume that the masonry there was absolutely impervious, whereas the masonry in the remainder of the dam was highly pervious. This seems to be an impossible condition.

Although somewhat outside the scope of this paper, the writer would like to invite attention to the so-called "economical profile" which, 50 or 75 years ago, was evolved by able engineers with mathematical inclinations. These profiles, based largely on maximum allowable pressures, are no longer of value except as traditions, yet they often appear in technical books as examples worthy of emulation. Attention is called to these obsolete profiles because they are unnecessarily confusing to the beginner.

Ice Pressure.—It is difficult to understand how quiescent ice produces material pressure on the back of a dam, except in certain sporadic cases which will be referred to subsequently. It appears that the basis of the somewhat common belief in such pressure came from knowledge of the destructive power of ice in motion. It is evident that an extensive area of sheet ice in motion would exert a pressure on the side of a river pier, which might approximate its crushing

Mr. Douglas. strength, which varies between 100 and 1 000 lb. per sq. in., depending on the purity of the water and the method of ice formation. It is on record that piers have been moved out of line and out of plumb, and that at least one bridge pier was raised bodily off its foundation, due to the adhesion of sheet ice in conjunction with a rise in the water level, but such destructive action has nothing to do with the pressure of quiescent ice.

Ice forms at a temperature of 32° Fahr., and, as the temperature falls, it shrinks and cracks. The cracks fill up with ice, in whole or part, and new ones are formed. Large areas of ice under a rise of temperature expand, pushing up into hummocks and thereby relieving the pressure, or, if the shores are sloping, it finds relief by sliding up the banks. At an overflowing spillway the water does not freeze against the dam. In a reservoir dam the ice freezes to the dam, and, if the water rises or falls after the ice is formed, the latter bends and breaks without much strain on the dam, as it is both brittle and low in tensile strength.

Recently, on a forebay dam, where there is a daily fluctuation in the water level, it was found, on examination, that the upper surface of the ice was 1 ft. thick; under it there was 1 ft. of water and under that 1½ ft. of ice. That was as far as the examination was carried, or at least as far as the writer has knowledge of it, and it is cited in the hope that others who are interested will supply additional information on the subject of pond ice, on which little has been written.

There are certain conditions under which quiescent ice is dangerous to the stability of a dam. If the ice is thick, and the adjacent banks are only a few hundred feet away and are vertical, or nearly so, the ice in expanding might exert material pressure, but even in this case it is doubtful if it would be great, unless the banks were of unyielding material, such as rock, and the water fluctuated in level. The Minneapolis dam failure offers a case of this kind. Ice several feet thick formed back of the dam, and the water was then drawn off. The ice sagged, forming an inverted arch with the dam for one abutment and the shore for the other. The water subsequently rose and through arch action pushed the dam out of plumb. If there are other records of sheet ice pushing dams out of normal, it would be interesting to have them recorded in the discussion of this paper. The writer recalls one or two cases in which it was a matter of doubt whether or not the ice was an active agent, but none seemed to point clearly to it as a direct cause of failure.

In regard to the often quoted 43 000 lb. per lin. ft. of dam specified by the Quaker Bridge Commission, it might be well if engineers would discontinue reference to this fact as having a bearing on the subject of general dam design. It is generally admitted that this pressure is conservative, and that at the time it was a wise precaution, but the writer does not believe it to be advisable to offer it now as a basis of dam design. Table 4 shows the effect of ice pressure on dam design, assuming 43 000 lb. per lin. ft.; and also the effect of uplift, assuming the full head at the heel and zero at the toe. Mr. Douglas.

TABLE 4.—Effect of Assumed Ice Pressure and Uplift on Dam Design.

The width of the base of the dam is given in feet.

Conditions.	Height of Dam.					
	5 ft.	10 ft.	30 ft.	60 ft.	100 ft.	250 ft.
Water pressure only	3.2	6.4	19	40	65	161
With ice pressure at 43 000 lb. per lin. ft.	41.6	41.9	45.8	56.7	76.7	167
With full uplift at heel, diminishing to zero at the toe	4.2	8.4	25	51	81	211
With above ice pressure and uplift	54.5	55.0	60	74	101	218

For simplicity of computation the top width of the dam is assumed to be 0.

Lindsay Duncan, M. Am. Soc. C. E. (by letter).—In February, 1906, the writer visited the Lake Cheesman Dam, of the Denver Union Water Company, with George T. Prince, M. Am. Soc. C. E., Chief Engineer, in order to observe the effect, if any, of the ice pressure on the dam. Mr. Duncan

At that time the reservoir level was 30 ft. below the spillway, and the ice was 12 in. thick. The days were warm and the nights cold, the maximum and minimum temperatures being about 50° and 20° Fahr., respectively, and the conditions, apparently, were favorable for ice thrust.

The writer was unable to detect any crushing or folding of the ice, but found that the portion directly against the masonry of the dam had softened and partly melted. It was evident that the masonry was at a temperature higher than the freezing point of water, and transmitted sufficient heat units to thaw the ice in contact.

This may possibly be the reason that engineers have no record of failure of a masonry dam on account of ice pressure.

Arthur P. Davis, M. Am. Soc. C. E. (by letter).—The main principles on which to base the design of a gravity dam for the resistance of uplift and ice pressure are given by the author with a conciseness and brevity rarely equalled. Mr. Davis.

For the reasons stated, the pressure of ice need seldom be considered; but, when there is a possibility that maximum ice pressure may be attained at the time the reservoir is full, it becomes important, because it is exerted near the top of the dam, at the time when both the water pressure and the uplift are also at their maximum, and

Mr. Davis. when low temperatures have a tendency to open vertical cracks and allow the water to enter the dam and thus obtain access to horizontal joints which might not otherwise be reached.

The determination of the perviousness of natural formations is one of the most difficult things in Nature. Any examination of such formations which disturbs them, changes the conditions which it is desired to know. For this reason, it is necessary to allow a large factor of safety in any estimates which involve this factor.

In general, it may be said that water will more readily follow seams or bedding planes than devious paths through the material of the rock. It follows that it will pass more readily and in larger volume in the direction of stratification than in a direction normal thereto. Similarly, stratified rock will permit percolation more easily and in greater volume than good, massive rock, such as granite.

Granular rock, such as sandstone, is likely to transmit more water through the rock itself than one of denser or finer grain, such as limestone or shale, but no exact rule of this nature can be laid down, because there are many varieties of each kind of rock, with various percolating capacities. In general, however, the following rules may be taken as a rough guide:

1. Massive or crystalline rocks, such as granite, gneiss and schists, will transmit water less freely than those of sedimentary origin.

2. Stratified rocks will transmit water much more readily in the direction of stratification than transverse thereto.

3. In the direction normal to stratification, sandstone will generally transmit water more readily than limestone, and the latter more readily than shale.

4. Stratification on a plane approximately horizontal is the worst possible condition for introducing upward pressures beneath a dam. Conversely, the most favorable position in this respect for stratified rock is in vertical beds.

An eminent geologist has stated that "it is safe to say that no foundation is entirely impervious." This may be too strong a statement, but at least it is unsafe to assume that any foundation is entirely impervious. If this is true, it follows that some provision for uplift should be made in the design of every masonry dam. This uplift may vary from a negligible quantity to the full hydrostatic head under an entire horizontal joint in the foundation. The amount of this force cannot possibly be foreseen with accuracy, and under ordinary circumstances cannot be foretold within rather wide limits; its estimation requires thorough investigation and the exercise of the highest degree of skill enlightened by the greatest available experience.

If the dam must be built as a purely gravity structure on a straight plan, the most economical method of meeting this problem is by increasing the batter on the water side of the original gravity structure, Mr. Davis.

such increase of batter to depend on the amount of uplift to be provided against. For dams of moderate height, the greatest safety with a given quantity of masonry is attained by a section roughly conforming to a right-angled triangle with the hypotenuse on the water slope. This form enlists the weight of the water to assist in holding the dam in place, and the increase of the batter may be carried to such a point that this resistance overbalances the tendency of the water to push the dam down stream.

The reason this principle is inapplicable in so many cases is that the average low masonry dam must serve as a spillway, and the impact of a large volume of water at the down-stream toe would be dangerous. Therefore, it becomes necessary to carry the masonry on such a slope as will prevent this impact and carry the water quietly away from the dam, allowing it to expend the accumulated energy in friction on the river bed at some distance below. This usually requires enough masonry to fulfill gravity requirements, without much batter on the back. The form described also has limits due to the height of the dam when the pressures at the down-stream toe approach the safe limits on the foundation. These limits vary widely with different foundations, and their determination in advance is so uncertain that a large factor of safety must often be allowed.

Where conditions permit, one of the surest and cheapest methods of providing the extra factor of safety required by the important and uncertain factors under consideration is to build the dam on a curve, arched up stream. If this be done, there is no possibility of its sliding or overturning without crushing either the masonry or its abutments. Any form of masonry is well adapted to the resistance of compressive strains, and it is on this that reliance should be placed when feasible.

The most frequent objection to such a proposition is that the compressive strains on the voussoirs of the arch and the rock of the abutments would be greater than safety would permit. Such a statement is usually based on the assumption that all the strains are taken by the arch and transmitted to the abutments. Such a result is absolutely impossible of attainment. It is impossible to deprive the dam of its weight; any properly built dam has resistance as a cantilever, irrespective of its plan, and no strains can be transmitted by the arch to the abutments until the resistance due to gravity and shear have been brought into play. The arch can only be made to take the residue, and if large strains are transmitted to the abutment this only confirms the necessity of the curve plan. If they are not so transmitted, and the dam resists all pressure by its weight, then the objection to the arch form is simply the increased cost.

To cite a concrete example, now fresh in the minds of all engineers: If the dam above Austin, Pa., had been built on even a

Mr. Davis. very slight curve, without any more masonry, it would be standing to-day. The length of this dam has been differently stated, but may be assumed as 400 ft. Had the dam been built on a circular plan of 400 ft. radius, with the same section, it would have contained about 5% more material and the pressure against it would have been about 5% greater, due to the greater surface exposed to water pressure.

The reservoir was filled in 1910 and stood for many hours in this condition, and, though it failed by cracking and sliding slightly on its base, the failure was neither sudden nor complete. This shows that, though the stresses were beyond its power of resistance, the excess was not great, and was probably less than 10 per cent. Let it be assumed that this excess was 10 per cent. If, therefore, these stresses had been increased by 5%, and the powers of resistance had been increased by 15%, the dam would have stood. Had there been doubt of the ability of the rock to take such a pressure, this could have been reduced to any desired amount by spreading the abutments and distributing the pressure over a larger area. That is, the stresses transmitted to the abutments would be only those above the resisting forces, considering the dam as a gravity structure.

Recent experience has shown the feasibility and efficacy, in some cases, of closing the crevices in the foundations wholly or partly by grouting them under pressure. This was accomplished successfully at moderate cost on the Ashokan Dam, and on several others of recent construction. The most striking instance of this kind which has come to the writer's attention is the Clackamas Dam, in Oregon, which was built on a foundation of semi-indurated volcanic ash, which was checkered in all directions by innumerable fissures, and, furthermore, was so soft that percolation was likely to cause destructive erosion. A triple line of holes was grouted along the up-stream toe of this dam, and recent information is that, since the dam has been in use, no perceptible percolation has taken place.

The effect of such grouting is not easy to foretell, and, like all other underground conditions, must be estimated with extreme caution.

Supplementary to this grouting process, a system of drains may be placed in the foundation, or in the masonry near the water face of the dam, which can be made to collect any waters percolating under high pressure and carry them harmlessly to the river bed below the dam.

An intelligent application of any or all of these remedies will make it unnecessary under any circumstances to provide otherwise for full upward pressure under any entire horizontal joint or plane.

It cannot be too strongly emphasized that no fixed rules, or "rule-of-thumb" method, can be adopted for the design of high masonry dams. Every dam of that kind is a problem unto itself, requiring the highest degree of skill and judgment for its correct solution; only the general principles and their nature can be set down in advance. Mr. Davis.

WILLIAM CAIN, M. AM. SOC. C. E. (by letter).—In the provisions for uplift presented in Cases 2 and 3, the author suggests that the upward pressure at the heel of the dam be taken as approximately equal to the static head. The writer suggests that this is too great an allowance. The uplift for a full static head, along the heel, could only occur, even approximately, where the foundation was composed of hard spheres, like marbles. In the case of gravel or earth, the pores are closed more or less with fine material, and the uplift is much less. Mr. Cain.

By experimenting with saturated sand, on a small scale, J. C. Meem,* M. Am. Soc. C. E., has proved that the water pressure on a given area, through sand having 40% voids, was about 40% of that due to the static head. Such a conclusion seems to be reasonable, and, if it should be confirmed by experiments on a large scale, for various materials, it would add greatly to our present knowledge. As Mr. Meem emphasizes, a gauge placed in the dam or foundation may register the full static head, but this does not prove that the full pressure is exerted over the whole foundation, but only over a portion, as his experiments indicate.

Mr. Wegmann has brought out the well-known fact that many high dams on good foundations, in the design of which no uplift was considered, have stood for years; and that, to allow for a full uplift in high dams, the dimensions would have to be increased beyond reason. Doubtless some uplift exists in all dams, but it is the writer's opinion that, at the heel, it never exceeds that due to half the static head, and is probably much less where a deep fill is placed against the up-stream face. It might then be supposed to decrease uniformly to zero at the toe, when the water which percolates there has free exit; but if it is under a head at the toe, then the uplift pressure there "will be equal to the head required to overcome the resistance to the water escaping at that point."

When saturated earth rests against the dam, on either face, it is a common error to find the weight per cubic foot of the earth in water, by subtracting from its weight in air the weight of a cubic foot of water. By this method, if the weight of earth in air is 100 lb. per cu. ft., the weight in water will be 100 — 62.5 = 37.5 lb. Assume that the earth has 40% voids; then 1 cu. ft. of earth contains 0.6 cu. ft. of solids, and the buoyant force of the water is the weight of an equal volume of water, or $0.6 \times 62.5 = 37.5$ lb.; hence the true weight of the earth in water is $100 - 37.5 = 62.5$ lb. per cu. ft., instead of 37.5 lb., as found by the erroneous method.

* *Transactions*, Am. Soc. C. E., Vol. LXX, pp. 365-372.

Mr. Cain. In computing the thrust due to the saturated earth against the dam, this weight, 62.5 lb. per cu. ft., for the earth, must be used if the full head of water is supposed to act on the dam. The full thrust on the dam will thus be the resultant of the earth thrust and the water thrust, as computed from the assumptions. It is certainly on the side of safety to proceed thus for the up-stream face, and, from our present lack of definite knowledge as to the modifying influence of the earth in diminishing the water thrust, the writer advises that this method be followed in designing.

If, however, future experiments should confirm Mr. Meem's small-scale experiments, the analysis would be as follows: Assume that an earth with 40% voids, transmits only 40% of the water pressure due to the full static head; then the water pressure on the dam from the surface of the saturated earth downward, will be only 40% of that due to the static head.

As regards the earth, a cubic foot with 40% of voids and having, therefore, only 0.6 cu. ft. of solids, will now be subjected to a buoyant force of only,

$$0.4 \times 0.6 \times 62.5 = 15 \text{ lb.};$$

hence, earth weighing 100 lb. per cu. ft., in air, will weigh 100 — 15 = 85 lb. in water. With this weight, the earth thrust is computed for the proper coefficient of friction of earth in water. A similar treatment would apply to any saturated filling on the down-stream face.

Recurring again to uplift, engineers until recently have omitted consideration of it in the design of dams, thus being on the side of danger. To counterbalance this, however, the full hydrostatic pressure has been supposed to act on that part of the dam which is in contact with the saturated filling (which sometimes extends to half the height of the dam), whereas there are good reasons for supposing that a much smaller pressure is exerted on this lower portion. From similar reasoning, it might be well to omit consideration of the pressure of any saturated filling on the down-stream face. It would seem that the amount of uplift should be taken in the increasing order for granite, sandstone, stratified rock with horizontal seams, earth, and gravel. In any case, cut-off walls and drains are desirable, especially in seamy rock where extensive grouting of crevices may also be required.

As the amount of uplift thus varies with different materials, and is an unknown quantity in any case, it should be left to the discretion of a competent engineer. It would be most unfortunate if a law was passed specifying a given uplift for all cases.

As to ice pressure, the writer thinks it should be allowed for in certain cases. In fact, as far as he knows, he was the first to suggest that allowance should be made for the influence of ice, floating bodies, etc.*

* *Engineering News*, June 23d, 1888.

It was also stated,* on the authority of Thomas C. Keefer, Past-President, Am. Soc. C. E., that "an ice bridge of about 90-ft. span, between two fixed abutments, expanded so from a rise of temperature, as to rise 3 ft. in the center." This afforded an opportunity to make a quantitative estimate of the ice thrust, and it was utilized at once with other data, in revising the cross-section of the proposed Quaker Bridge Dam by the Board of Engineers. As is well known, the design recommended by that Board, meritorious as it was, and in advance of any previous design, was not adopted finally. Mr. Cain.

The writer is glad to note that both ice pressure and uplift have been considered in the Kensico and Olive Bridge Dams. It remains now to go further and, by experiments on a large scale, determine, more closely than hitherto, the amount of this uplift and the allied subject, the hydrostatic pressure of ground-water.

In conclusion, the writer's thanks are due to the author for his timely paper and for the fair and comprehensive manner in which he has presented the main points at issue.

J. W. LEDOUX, M. AM. SOC. C. E. (by letter).—This is a very important subject, and one on which pertinent facts and the judgment of engineers with large experience in this particular line are valuable. Unfortunately, however, there is evidence that some of those who have taken part in the discussion have not had this broad experience, as their judgment appears to be erroneous. Mr. Ledoux.

It is not by any means certain that the failure of the dam at Austin, Pa., was due to upward pressure. It is quite probable that if upward pressure were excluded entirely, the dam would have failed in exactly the same manner. It is a question of the coefficient of friction. The dam did not go down into rock, but only rested on or near the surface. These rock layers were nearly horizontal, from 2 to 6 in. or more in thickness, and parted by unctuous clay. The coefficient of friction between two surfaces of rock separated by clay or soft, wet shale may not be more than from 0.3 to 0.5, while the dam at Austin would probably have failed if the coefficient had been 0.55.

It is a mistake to become hysterical about upward pressure, and there is a possibility that it is of minor consequence in dams in which the design and construction are carried out in a reasonable way. Some one has made experiments recently which convince him that upward pressure is exerted only in the voids in the material. In other words, if a dam rested on sand having only 40% of voids, the maximum possible upward pressure would be 40% of the bottom area of the dam. If a dam rests on rock, and the concrete or masonry is reasonably constructed with flush mortar, the chances are that the voids are far less than 40%, and if this be considered and the fact that

* *Engineering News*, June 30th, 1888.

Mr. Ledoux. the maximum pressure is greatest at the up-stream toe of the dam and zero at the down-stream toe, the total upward pressure under the dam could not exceed 0.2 of the hydrostatic pressure. However, if the rock foundation contains a horizontal crack a short distance below the base of the dam, the upward pressure might be considerably greater.

If a dam is built of good concrete or masonry with good Portland cement mortar, and the design is ample, there is no danger of defects occurring in the structure itself which will cause failure, such as temperature cracks or cracks due to placing new work on work which has been finished for a considerable time. These should be provided against as far as possible, but, in the writer's experience, they have never been known in themselves to cause the failure of a dam or retaining wall. If a masonry or concrete dam without reinforcement has been finished in warm weather, it is almost certain that vertical transverse cracks, from top to bottom, will occur as soon as very cold weather comes. These are large enough at times to permit considerable seepage of water. Before the winter is over, however, these cracks close up, the leakage disappears, and in the next season they can hardly be found. The filling is probably an efflorescence of magnesia or lime from the cement; therefore, these temperature cracks are not of sufficient consequence to warrant the use of expansion joints, and, besides, the quantity of reinforcement required to prevent them would be extremely large, and the results obtained would not warrant the expenditure. If a dam is built of concrete and contains a very large proportion of boulders or heavy rocks of good quality, it is possible that these cracks will not occur, but this opinion is not based on a sufficient number of cases to be absolutely reliable. In building large dams, it is necessary to depend on all kinds of labor, and it is not possible to have the same kind of expert labor on the concrete as is constantly employed by the best sidewalk cement paving companies. If it were, it might be possible to build a dam which would not crack or check on account of changes of temperature.

No engineer of experience cares to build a masonry dam at all unless he can found it on a material which has sufficient bearing power. The limit in this respect might be considered shale rock, which would bear safely at least 10 tons per sq. ft. Such a rock, of course, is not capable of withstanding erosion due to falling water, therefore, means must be supplied to prevent overtopping the dam in floods. In other words, an independent spillway must be provided. Where this is impossible and it is necessary for the water in floods to pass over the center of the dam, if the material on which the dam is founded will not resist erosion, the only thing to do is to provide an apron of heavy stone laid in mortar and 10 ft. or more in thickness; the upper surface of this apron should be below the water level and

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be carried down stream 20 ft. or more, depending on the height of the dam.

To prevent water from passing under the dam, a cut-off trench must be sunk to reasonably impervious material. Such a trench need not be more than 6 ft. wide, and should be under the up-stream toe. Of course, it is filled with concrete or masonry and thoroughly incorporated with the main structure of the dam, but the main portion of the dam should go deep enough to have a sufficient barrier of natural material to prevent it from sliding. In most cases 10 ft. in rock will be sufficient for this, but if the dam is very high a greater depth becomes necessary. Down stream from this trench the writer believes it would be good practice to place longitudinal open drains and connect them at frequent intervals with transverse drains extending down stream, so as to eliminate as far as possible the effects of upward pressure. This can be done without any material increase in cost, and is far better than to assume an excessive upward pressure and then build the dam of sufficient section to resist it.

To provide against a possibility of trouble during construction, English engineers usually leave, in the lower part of the dam, an opening sufficiently large to carry away any possible floods. Concrete a week old will stand for a short time a large depth of water running over it without danger of destructive erosion, so that excessive precaution in this respect seems to be unwarranted, except in a case where the water flowing over the dam would erode the material from the down-stream toe. These openings are usually circular in form, and are recessed so that the masonry or concrete finally placed will dovetail in with the remainder of the work.

The writer never had much apprehension concerning ice pressure. If the expanse of ice is very great, it would appear that no material pressure could be exerted without crushing the ice, due to the effect acting on a long column. The main danger is with the coping, where water may freeze in horizontal cracks and lift the stones. Of course, this can be obviated by putting heavy steel bolts through the coping, embedding them in the masonry to a depth of 4 to 5 ft. Where the ends of the dam are close to vertical rock cliffs, the danger of destruction at these points, due to ice pressure, becomes very much greater, because the ice has no chance to slide up along the ground, and the distance is so short that the column weakness is not material. This consideration, however, is theoretical rather than practical, as the writer has never seen any failure due to this cause. There are hundreds of stand-pipes in northern latitudes in America, where ice freezes to a thickness of 2 ft. or more, and yet failures from ice expansion are comparatively rare.

The top thickness of the dam, in the writer's judgment, should never be less than 4 ft., and 6 ft. or more is better, depending on the

Mr. Ledoux. size of the stream and the height of the dam. This, however, is not capable of exact determination based on calculation, and in each case must be left to the judgment and experience of the designer.

It is unnecessary to discuss the theoretical considerations or design of the main section of the dam. These have been worked out most elaborately and carefully by Messrs. Wegmann, Gregory, Brodie, and others. When all the external forces are known, it is not difficult to design the dam with the most economical section to resist these forces safely. The main difficulty, however, is with the assumed data. Some of the sections which have been considered contain nearly twice as much masonry as others. If the smallest section is perfectly safe and can be built for several hundred thousand dollars less than the larger one, it is, of course, bad engineering to build the larger section. Where there is a doubt, and a mistake would cause loss of life as well as property, it is the absolute duty of the engineer to be on the safe side, but this then becomes a question of experience. One engineer, with abundant experience under all kinds of conditions, will make a design in which the safety is in his mind beyond question. Another engineer, however, who has read all these discussions, and who has not himself had this broad experience, would not dare to be responsible for such a design. Therefore, the question is a very serious one, and no simple solution appears to be possible.

Mr. Le Conte. L. J. Le Conte, M. Am. Soc. C. E. (by letter).—A survey for the purpose of determining the character of the foundation on which a dam is to be built is highly necessary in all cases; but too much faith should not be placed in the results thus obtained. The reason is plain and practical. Suppose the survey indicates favorable conditions, the question at once arises: "How do you know that these favorable conditions will remain so *ad infinitum?*" The experience of the big concrete dam near San Mateo, Cal., built by the Spring Valley Water Company, of San Francisco, in 1888, is cited as an example. In the writer's opinion, this splendid structure is a lasting monument to common sense and sound engineering judgment. The foundation was stripped, examined carefully, and found to consist of a very fair quality of argillo—calcareous sandstone—full of seams and fissures, but all these were well filled with calcareous cement. The sandstone, of course, was porous, and, when exposed to water, soon became saturated. The concrete dam was built of the same sandstone crushed to small sizes and, of course, it was also porous. These facts being well-established, the engineer in charge, Mr. Herman Schussler, gave the dam such a cross-section as would stand with safety the water pressure from the reservoir as well as uplift on the entire base. This extremely safe load covers everything to be considered except ice pressure, but ice never causes trouble at and near San Francisco.

Mr. Le Conte.

This dam had its severest test during the great earthquake of April 18th, 1906, which, together with the conflagration which followed in its wake, destroyed nearly half of San Francisco. The enormous ground fissure or geological fault passed through the chain of lakes built by the Spring Valley Water-Works, and, as luck would have it, passed close to the up-stream toe of the dam, cutting off the tunnel which delivered the main water supply to San Francisco. The earthquake cracked and shattered the bed-rock below the base of the dam, and gave rise to numerous new springs in the creek bed below the dam site; but, with the lapse of time, these have choked up and now discharge an insignificant quantity of water. This accident shows that even if one were absolutely sure of a perfect bed-rock, one could not depend on it in an earthquake. This dam, with 30 000 000 000 gal. of water behind it, stood the earthquake shock perfectly, and the writer believes that this remarkable result was due to its generous proportions and masterly construction.

The attention of practical engineers is called to the constant and ever-present danger to which the constructing engineer is always exposed and against which he cannot protect himself by any possible means, namely, imperfect materials and imperfect workmanship. These dangers are mentioned because of the unexpected results experienced with this great concrete dam, which are certainly worthy of record. It was not built with concrete *en masse,* but, on the contrary, in separate blocks of artificial stone, say, from 50 to 100 tons each, moulded in place. These blocks were interlocked and dovetailed together in such a way that it was confidently expected there would not be at any place a continuous seam or joint through the dam. The top of each constructed block in each layer varied in height so as to make broken lines, and each block was located so as to break joints with the lower and upper layers. In addition, in each block were placed twisted iron rods with their ends sticking out to tie into neighboring blocks.

In spite of all this care and precaution, when the lake was filled with water, to the surprise of every one, a leak was discovered 90 ft. below the crest of the dam—where the concrete, from front to back, was nearly 100 ft. thick. This leak was not a simple "weeping," but, on the contrary, squirted out a considerable distance. It is proper to state, however, that it finally grew less and less, and ultimately became a mere "weep," overgrown with algæ, making a green spot on the back of the dam.

This leak, of course, was unimportant, but it serves to show how utterly absurd it is to assume that a masonry dam can be made absolutely water-tight. The writer believes that such perfection is wholly impracticable, if not impossible.

Mr. Le Conte. In regard to the curvature of the dam in plan: Mr. Schussler states: The cross-section of the dam would have a factor of safety of 5 to 1 if it were built straight across the valley, some 660 ft.; but it is on a curve up stream, the arch having a rise up stream of one-eighth of the length, and the center line a radius of 724 ft. The factor of safety of the dam is greatly increased by curvature, and it will stand safely twice the pressure contemplated. The sound wisdom of this conclusion was amply tested by the earthquake of 1906; and, inasmuch as such shocks, of greater or less intensity, are to be expected in every State in the Union, they cannot be left out of consideration in the design of any dam of importance, where the lives and safety of many citizens are constantly in peril.

The cross-section of the dam was based on the following assumptions:

Total concrete masonry....................	1 329 tons.
Less gross uplift on base..................	1 050 "
Unbalanced weight.................	279 tons.
Total water pressure from the reservoir, at right angles to the face...............	478 tons.

Both these forces act at a point on a vertical line through the center of gravity of the dam. The resultant force thus developed cuts the base of the dam just inside of the middle third, the angle which the resultant makes with the vertical at that point being about 50 degrees. Accordingly, the base, resting on bed-rock, is sloped downward toward the up-stream side, 10 ft. in 191 ft., which is the width of the base. This sloping bed-plane is broken by four vertical step jogs, of 2.5 ft each. This makes a rock trench, 191 ft. wide, 10 ft. deep at the lower end, and 20 ft. deep at the up-stream end, which secures the dam against the possibility of sliding on its base, because of the very oblique resultant.

This assumption of a full hydrostatic pressure over the entire base of the dam, at first glance, seems to be unreasonable; but, in view of the uncertainty of the great ruling factors involved, the final summation of which cannot be predicted with anything like exactness, the assumption at once becomes conservative and entirely rational.

In addition to all this precaution, the dam in plan also has a curvature up stream which adds greatly to its stability and, at the same time, guards against the possibility of vertical fissures opening on the back of the dam because of the elasticity of the concrete masonry and the bed-rock itself.

The universality of the uplift over the entire base of a dam is shown by the history of the construction of the Assuan Dam across the Nile.* The only absolutely safe and sure course is to assume a

* *Minutes of Proceedings*, Inst. C. E., Vol. CLII. 1902–03.

full hydrostatic head over the entire base of the dam, then depress the up-stream toe so that it will be at least 6% deeper than the down-stream toe, and then terrace the foundation bed into four or five steps facing up stream to guard against sliding on the base. Some engineers also build a cut-off wall under the foundation bed. The propriety of this proceeding should be determined by test borings. Mr. Le Conte.

In conclusion, a comparison of the cross-sections of standard dams designed by Krantz and Professor Rankine for the same height, 185 ft., is interesting:

Krantz Dam..........................	12 015 sq. ft.
Rankine's Dam......................	12 728 " "
Schussler's Dam....................	18 970 " "

The writer thinks that the generous section adopted by Mr. Schussler has everything to commend it.

C. L. Harrison, M. Am. Soc. C. E. (by letter).—After the failure of the dam at Austin, Pa., several engineers were discussing the effect of uplift and ice pressure on dams, and it was finally suggested that a memorandum on this subject be prepared and presented to the Society for discussion. It is gratifying to note that so many valuable discussions have been presented. Mr. Harrison.

As mentioned by Mr. Hazen and others, the late James B. Francis, Past-President, Am. Soc. C. E., presented to the Society, on May 16th, 1888, a masterly discussion on "High Walls or Dams to Resist the Pressure of Water," and deduced formulas, which he called skeletons or diagrams, to which additions could be made to provide for ice pressure, wave action, uplift, and all other contingencies to which the wall might be subjected. The present discussion may be considered as a continuation of this paper, in that an effort is made to determine what provision shall be made for uplift and ice pressure.

Ice Pressure.

This subject has been discussed by Messrs. Cole, Gerry, Wegmann, Gregory, Brodie, Douglas, Duncan, and Davis.

It is generally agreed that this force should be considered, and allowance made in some cases, but not in others. The limitations of these cases are not definitely stated by those who have discussed the subject. In order to give the matter more definite shape, it may be suggested that, under the following conditions, it is not necessary to provide for ice pressure:

1.—For the ordinary storage reservoir with sloping banks, in climates where the maximum thickness of ice is 6 in. or less—for dams with southern exposure this limit may be placed as high as 1 ft. None of the discussions fixes this limit, but it is what the writer has in mind as a reasonable provision.

Mr. Harrison.

2.—For reservoirs which are filled during the flood season and from which all the stored water is drawn off each year during the low-water season. This would include even the large reservoirs on the head-waters of the Mississippi River, where the ice has a thickness of more than 4 ft., and the atmospheric temperatures reach 50° below zero.

3.—For storage reservoirs where the water will be drawn off each year during the winter to a level where the dam is strong enough to resist the ice pressure.

4.—For reservoirs where the contour of the ground at the high-water level is such that the expansive force of the ice will not reach the dam.

Mr. Gerry thinks the ice pressure, where acting, should always be allowed for, up to the crushing strength of the ice, without stating what he would use as the crushing strength of ice, while Mr. Douglas is inclined to make no allowance for it except in rare cases. The views of the others are rather indefinite, but are generally between these two.

The dams cited in Table 5, where a value has been given to the ice pressure in the design, are in the vicinity of New York or Boston, where the maximum thickness of ice may be taken as about 2 ft. No allowance is made for ice pressure in the New Croton Dam, which is in the same climate.

TABLE 5.

Dam.	Location.	Allowance for ice pressure, in pounds per linear foot.
Wachusett	Boston	47 000
Olive Bridge	New York	47 000
Kensico	"	47 000
Croton Falls	"	30 000
Cross River	"	24 000
New Croton	"	No allowance.

The stored waters in all the reservoirs in Table 5 are for domestic supply, and, excepting Olive Bridge and Kensico, are in service. The reasons given for the smaller allowances made for the Croton Falls and Cross River Dams are that local conditions will prevent the full ice thrust from reaching the dams, and also that they are located up stream from the New Croton. If either of these dams should fail, no valuable property would be damaged, and the waters would flow into the New Croton Reservoir. Those responsible for the design of the New Croton Dam believed that no allowance should be made at this dam for ice thrust. At first glance, this looks like a wide range in judgment, but it must be remembered that the foregoing statement gives only a part of the facts, and to this must be added the local conditions and the service the dam is to render in each case before judgment is passed on the wisdom of the design. Mr. Harrison.

Reservoirs for domestic supplies are generally drawn down during the ice period, and the greatest expansion of the ice occurs at the end of this period, thus applying the pressure at a point below high-water level, where the dam is strong enough to resist it. If, however, such reservoirs are to be at high-water level during, and especially at the end of, the ice period, at any time during their service, then the proper allowance should be made for ice pressure in the design. The daily fluctuations in the water level in the forebay at power dams will usually prevent the ice from freezing to the dam, which, therefore, will not be subjected to thrust caused by the expansion of the ice in the pool above the dam. In such cases, the proper course seems to be, not to reduce the allowance, but to omit it altogether. If, however, a storage reservoir is to be filled to the high-water level during the full ice period, at any time during the life of its service, then not a partial, but the full ice pressure should be allowed for in the design of the dam. It is contemplated that the Kensico Reservoir is to be kept at or near the high-water level at all times, and therefore will be subject to full ice pressure at high-water level; also, the Olive Bridge and Wachusett Dams may at intervals be subject to this pressure at high-water level. It is entirely possible that it would be proper to allow for ice pressure on a dam in a given locality and also proper to make no allowance for such pressure on another dam in the same locality, depending on the service each is to render.

The fact that so many dams have been designed and built without making a specific or separate allowance for ice thrust, and have for years stood the test of actual service without failing, is an indication that ice pressures may not be as great as sometimes thought, or that the factors of safety allowed for other purposes are sufficient to take this pressure. On the other hand, in the cases mentioned in this discussion, there seems to be good and sufficient reason for allowing for ice pressure in the designs.

Uplift.

Each of the twenty discussions presented on this subject recognizes the existence of uplift and that it should be considered, and, when an important factor, should be allowed for in the design of the dam. Two methods of doing this are suggested, namely, by drainage, and by increased section. It is also agreed that the uplift in the foundations and in the dam itself should be considered separately.

Uplift in the Foundations.—Take the case where the foundation rock, as found in Nature, is stratified horizontally, with well-defined seams or beds. In what manner does the water enter these seams, and how is the upward pressure produced? Suppose there is such a

Mr. Harrison. seam 10 ft. below the surface of the rock. The upper ledge is resting on the ledge immediately below it; the two ledges are in contact for a part of the area considered, and they are not in contact in other parts of this area. If water is introduced into this seam under a low head, it will flow into and fill the areas which are not in contact, and, if the water has no outlet, a pressure will be produced over this partial area; but this pressure will not exist over that part of the area where the ledges are in contact. If the head of water be increased, and its free connection with the water in the seam is maintained, the pressure will be increased over this area; if this head be now increased, so that its uplifting force acting on this partial area is greater than the weight of the upper ledge and the other material resting on it, then the upper ledge will be lifted, so that there will be no points of contact in the two ledges, the pressure will act over the entire area, and the upper ledge will be floating. Suppose, now, a dam be substituted for the material resting on the bed-rock, and the reservoir be filled, then the pressure will still act over that part of the seam where the two ledges are not in contact, and cannot act over the whole area until the pressure acting over this partial area is sufficient to lift the upper ledges and also the dam. The dam would probably slide before it would be entirely afloat. The section of any dam should be made heavy enough to prevent it from being thus lifted.

Under the conditions which usually exist, the intensity of pressure at the heel of the dam can never be more than that due to the static head, but it may be less if the water enters the seam some distance up stream from the dam, or is otherwise obstructed at the point of entrance, as mentioned by Professor Cain. The pressures at the toe of the dam will be as stated in Cases 2 and 3 of the paper. As already pointed out, this pressure will act over only a part of the area. In designing the Olive Bridge and Kensico Dams, Messrs. Gregory and Brodie allowed for its acting on two-thirds of the area, while Messrs. Meem and Cain think it acts on less than half the area, and that it would not act over the entire area if the dam were founded on other material than rock.

If we consider the foundation to be more or less broken up, but not horizontally stratified, then the pressure would act over an area in some proportion to the total area of the seams, which would generally be much less than in the case of stratified rock. These same principles can be used in determining the allowance for uplift in Case 3.

Mr. Gerry thinks it impossible to make a water-tight joint between the bed-rock, as described in Case 1, and the masonry above. Still, such joints have been built practically water-tight in a number of masonry structures, but this result is more difficult of attainment in concrete structures.

Mr. Harrison.

To reduce or prevent the uplift, it is generally agreed that cut-off walls should be built near the heel of the dam, and that under-drains should be provided between the down-stream face of the cut-off wall and the toe of the dam. Some of the discussions suggest that the section of the dam should be heavy enough to counteract the uplift, without drainage. It is possible that the conditions may indicate the use of one method in some locations and another in other locations. Neither the cut-off wall nor drainage will wholly prevent the uplift much below their location, in the case of horizontally stratified rock, as there may be an open seam which will admit the water under pressure just below the ledge on which the cut-off wall rests. In deep foundations complete under-drainage is practically impossible.

Uplift in the Masonry.—It is agreed that in the case of horizontal cracks or seams there will be an upward pressure when the water enters them, and allowance should be made for it, either by drainage or additional weight of masonry. Both are used in the Olive Bridge and Kensico Dams. In case additional weight is used to counterbalance this force, then the allowance made should bear some relation to the character of the masonry used and the climatic conditions under which it is laid.

Great care should be taken in building the dam to reduce these horizontal joints to a minimum. In concrete masonry, they cannot be entirely avoided when the conditions require intermittent work, especially suspension during the winter. In rubble masonry, they can be reduced so as not to be a serious matter, and under some conditions can be entirely eliminated; but, in concrete work, it is very difficult if not impossible to join new work to old and make a water-tight joint. One cause of this may be that heat is developed while the concrete is setting. In massive work, the temperature of the concrete frequently reaches 100° and 105°, remains at these temperatures several days, and then cools off gradually. The old concrete on which it is placed is evidently at a lower temperature. These conditions may prevent the bonding of the new work to the old. Also, concrete is usually deposited wet, resulting in the surface being practically level, and, when work is suspended, on account of cold weather or otherwise, the surface is not only level but water containing more or less of the wash from the concrete may be left standing over the surface, as mentioned by Mr. Barnes. Provision should be made, in the contract and in the supervision of the construction, for removing such deposits and thoroughly cleaning the surface before resuming concrete work. Both the specifications and the supervision should regulate the wetness of the mixture. For both concrete and rubble masonry, the specifications should state definitely how new work is to be joined to old, and also how the junction between the masonry and rock is to be made. Provision to pay for this as a separate item may be advisable.

Mr. Harrison.

For masonry dams, the up-stream face stones should be laid in a heavy bed of rich mortar, and all vertical joints should be thoroughly filled with the same mortar. Throughout the entire area of the dam the large stones should also be set in a heavy bed of mortar. All surfaces should be thoroughly cleaned immediately before the mortar is placed on them. When filling in between the larger stones, mortar should first be put in the spaces and the smaller stones then worked down into it. The masonry should not be leveled up at any time during the work. Experience has shown that practically water-tight work will result when these precautions are faithfully carried out.

The Wachusett, Olive Bridge, and Kensico Dams have the heaviest sections of any of the dams referred to in the discussions. These result from the allowance made for ice pressure and uplift. They are all located immediately above towns and valuable property, and their failure would result in great property loss as well as loss of life. Still, it is difficult to imagine the failure of any dam of the class under consideration without disastrous results. The discussions by Messrs. Gregory and Brodie are especially interesting in that they give the precautions taken to meet both the uplift and ice pressure in two of the most recently constructed large dams. As they have been designed and approved by engineering talent of such high standing, they may be taken as the best practice at this time for conditions similar to those at Olive Bridge and Kensico.

As stated by Mr. Davis and others, no general rule can be laid down to allow for uplift and ice pressure, which will be applicable to all cases. A special investigation should be made at each site, by borings, test-pits, and otherwise, in order to determine as nearly as possible the exact conditions as to the nature of the foundation rock and the character of the seams and cracks in it, as well as to obtain all other information which will lead to the proper diagnosis of the case. The conditions revealed when the foundation is uncovered and can be fully inspected may not be the same as indicated by the preliminary investigations, and may suggest some modification of the original design.

An analysis of the discussion indicates the following conclusions:

1.—For any stable dam the uplift in the foundation cannot act over the entire area of any horizontal seam, and in the masonry it cannot act over the entire area of any horizontal joint.

2.—The intensity of uplift at the heel of the dam can never be more, and is generally less, than that due to the static head. Also, this uplift decreases in intensity from the heel to the toe of the dam, where it will be zero if the water escapes freely, and will be that due to the static head if the water is trapped.

3.—The uplift in the foundation should be minimized by a cut-off wall, under-drainage, and grouting when applicable; and in the dam itself by using good materials and workmanship, and by drainage when advisable. Mr. Harrison

4.—The design should be based on the conditions found to exist at each site after a thorough investigation by borings, test-pits, and otherwise, and modified if found necessary after bed-rock is uncovered.

After all, determining the proper allowance to be made for the uplift and ice pressure is not an "exact science," but the engineer, after considering all the obtainable facts bearing on the case, must use his best judgment, based on experience and observation, in making these allowances; and the final design should be made so as not to involve undue risks on the one hand, or unnecessary cost on the other.

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TRANSACTIONS

Paper No. 1222.

THE PROBLEM OF THE LOWER WEST SIDE MANHATTAN WATER-FRONT OF THE PORT OF NEW YORK.*

By B. F. Cresson, Jr., M. Am. Soc. C. E.

With Discussion by Messrs. S. W. Hoag, Jr., H. McL. Harding, James Forgie, Edlow W. Harrison, Charles H. Higgins, M. Lewinson, T. Kennard Thomson, Calvin Tomkins, Ernest C. Moore, Reginald Pelham Bolton, Henry B. Seaman, J. H. Gandolfo, E. De V. Tompkins, A. W. Robinson, Augustus Smith, and B. F. Cresson, Jr.

Many municipal problems confront New York at the present time, and none is more pressing than that of the organization of the lower Manhattan water-front on the North River. The present congestion of growing business, the demand from all classes of carriers for additional water-front facilities, the coming of larger passenger steamships, and the generally unrelated use of this water-front, demand immediate consideration of the conditions, and the formulation of a policy toward which reorganization should grow.

The lack of public interest and knowledge of conditions, together with the trade rivalries of the carriers, make it most difficult to determine on a plan and policy. The problem largely involves economic conditions and commercial necessities, and, in this paper,

* Presented at the meeting of February 21st, 1912.

an endeavor will be made to describe briefly these conditions and necessities and indicate in general terms a plan for relief.

Much has been written concerning the adequacy of the terminal facilities and the excellence of the administration and control of the greater seaports of Northern Europe, and, in formulating a plan and policy for New York, what has been done in those seaports should be carefully studied.

The harbor of the Port of New York is not lacking in natural advantages, and, in comparing it with the important harbors of Northern Europe, one of the impressive features is the available space for almost indefinite expansion, provided it is properly organized. Situated directly at the entrance to the sea, it has an excellent channel, is amply protected from the sea by the comparatively narrow passageway between Fort Hamilton and Fort Wadsworth; and the wide expanse of the upper bay, which contains much shallow water, is capable of being enlarged for commerce by dredging and by the construction of piers or basins.

The tidal fluctuation is not great enough to interfere in any way with navigation from the open sea or from inland waters. The rivers are sufficiently wide to permit piers to be constructed at right angles, thus insuring economy in the use of the water-front by berthing ships end on, and not (as in many ports) requiring them to be docked against bulkhead walls and thus taking up a greater extent of water-front.

Greater New York has a water-front of about 450 miles, and if to this is added that of the New Jersey shore, including also Newark Bay, which are important parts of the port, it will easily be seen that it has a harbor which offers a wealth of opportunity.

At many European ports, practically all the water-front has been dredged from the mainland at great expense. It has been said that the things which come naturally are usually less prized and conserved than those which are obtained only by great labor; and the extravagant use of some parts of the New York water-front affords a remarkable contrast to the care and foresight with which European port authorities have organized and conserved the frontage which has been created for the most part artificially.

Properly controlled and organized, there is almost unlimited opportunity at New York for commerce to grow, whereas, in European ports generally, additional room can be obtained only by dredging farther inland, and it appears that there must be a limit to this growth owing to the congestion which already exists in the comparatively narrow rivers leading to the sea.

In many European ports, the tidal fluctuation is so great as to make wet docks a necessity, that is, the port itself is constructed back from the river, access thereto being through locks, and usually only at high water. This causes very expensive construction and more or less complicated operation.

New York is already the greatest port in the world, in its volume of commerce, and when the Panama Canal and the New York State Barge Canal are opened, a still greater volume will naturally seek accommodation there. Being the natural meeting point of both rail and water transportation lines, facilities for the interchange of freight and passenger business on a very large scale must be anticipated.

The responsibilities of New York as the principal port of entry of North America are very great. Liverpool is the home port of the Cunard and White Star Lines, London of the Atlantic Transport Line, Havre of the French Line, Antwerp of the Red Star Line, Rotterdam of the Holland-America Line, Bremen of the North German Lloyd, and Hamburg of the Hamburg-American Line. While, of course, these great companies send ships to all parts of the world, their very best and largest vessels come to New York, and there it is the duty of the authorities to provide facilities which will give as much convenience as possible.

Practically every important railroad in North America has a terminal in New York. Those not having their rails physically either in New York or New Jersey, have as their connecting links the coastwise steamships.

If the supremacy of New York as a port of entry for the United States for passengers and high-class freight is to be maintained, there must be proper organization in order to increase its capacity as a port of entry and exchange for coarse freight, and make provision for industrial as well as commercial development.

Very little has been done constructively in the last few years, but certain conditions have now arisen which make prompt action necessary. At present there is congestion at only one section of the water-

PLATE VI.
TRANS. AM. SOC. CIV. ENGRS.
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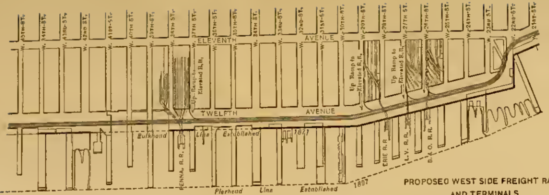
CRESSON ON
WATER-FRONT FACILITIES,
NORTH RIVER, NEW YORK CITY.



PROPOSED WEST SIDE FREIGHT RAILROAD
AND TERMINALS
SHOWING CONNECTIONS FROM EXISTING YARDS
TO PROPOSED ELEVATED FREIGHT RAILROAD

POSSIBLE INSTALLATION OF LONG PIERS
BETWEEN CHAMBERS AND DESBROSSES STREETS,
NORTH RIVER

SCALE OF FEET
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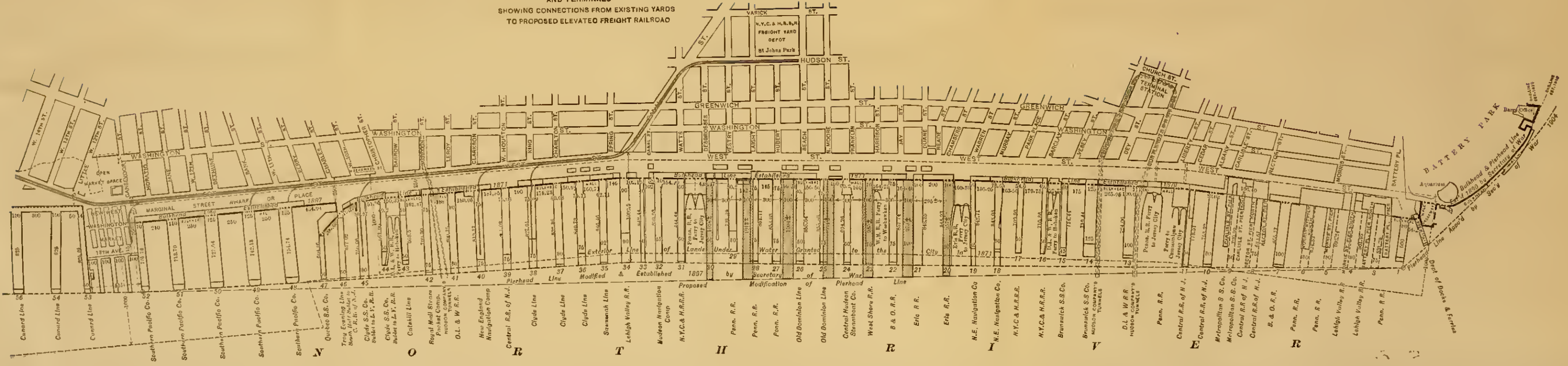


front, and this is at the most desirable locality, namely, along the west side of lower Manhattan.

Presentation of the Problem.

There are a number of factors which make the problem of the reorganization of the lower Manhattan water-front difficult, and it is necessary to take them all into consideration to determine intelligently on the plan which best meets all conditions. The principal factors may be stated as follows:

1.—The desirability of docking the large transatlantic steamers at the lower Manhattan water-front;

2.—Provision for longer steamships;

3.—The railroad occupation, and its already congested condition;

4.—The dangerous, inadequate, all-rail connection of the New York Central Railroad;

5.—Congestion;

6.—The general increase in the commerce of the port.

All these factors are involved in the problem, and it must be looked at, not from the point of view of any individual carrier, but from that of the commercial welfare of the City and of the country as well.

The business interests using the water-front are shown by Table 1, the large proportion occupied by the railroads being evident.

TABLE 1.—Business Interests Using the New York Water-Front.

	From north side of Pier new 1 to 125 ft. south of Pier new 48, 11 780 ft. = 2.23 miles.	From north side of Pier new 1 to north side of West 30th Street, 20 658 ft. = 3.91 miles.
Transatlantic steamships	1.4%	17.5%
Coastwise steamships	15.6%	24.3%
Railroads	47.9%	30.8%
Hudson River boats	5.3%	3.0%
Sound steamers	10.0%	5.7%
Ferries	9.5%	7.8%
Open wharfage	4.3%	3.9%
Miscellaneous: coal, ice, dumps, oysters	5.8%	6.9%
Recreation piers	0.2%	0.1%

Passenger Steamship Terminals.

As a terminal for the transatlantic service for passengers and some high-class freight, no part of the harbor is as desirable or as convenient as the lower Manhattan section, because of its proximity to the sea and the fact that it is in the heart of the business, financial, railroad, and hotel districts. The traveler coming directly to Manhattan does not have the inconvenience and difficulty of transferring from some other part of the harbor to the central district, and it is to this district that practically all travelers desire to come. Of course, long piers could be built in South Brooklyn and Staten Island to accommodate large passenger steamships, but, as an officer of one of the transatlantic lines has emphatically stated, the same reasons which impelled the New York Central and Pennsylvania Railroads to spend millions in establishing stations in the center of Manhattan make it desirable that the leading steamship lines have termini as near as possible to the same district. The desirability of retaining this business in Manhattan, therefore, is manifest because of the mutual advantages to the City and the steamship lines, and applications are on file at the Dock Department for many additional piers for steamship service.

Terminals for Ships of the "Olympic" Class.

The necessity of providing accommodation for the larger steamships now building for this port is very great. When the S. S. *Olympic* was nearing completion, an application was made to the Secretary of War for the extension of the pierhead line in lower Manhattan, so that the new Chelsea steamship piers between 12th and 23d Streets might be increased by 100 ft., making them 925 ft. long, in order to accommodate this vessel, which has a length of 882½ ft. There is no pier in lower Manhattan long enough to accommodate this ship, except by extending the pier farther into the river or by dredging inland. It was only after the urgency of the immediate requirements had been shown that temporary permission was given for two years by the Secretary of War for the extension of two of the Chelsea piers, and this was done with the distinct understanding that within that period the City must commit itself to a plan which would make provision for long ships, without permanent encroachment on

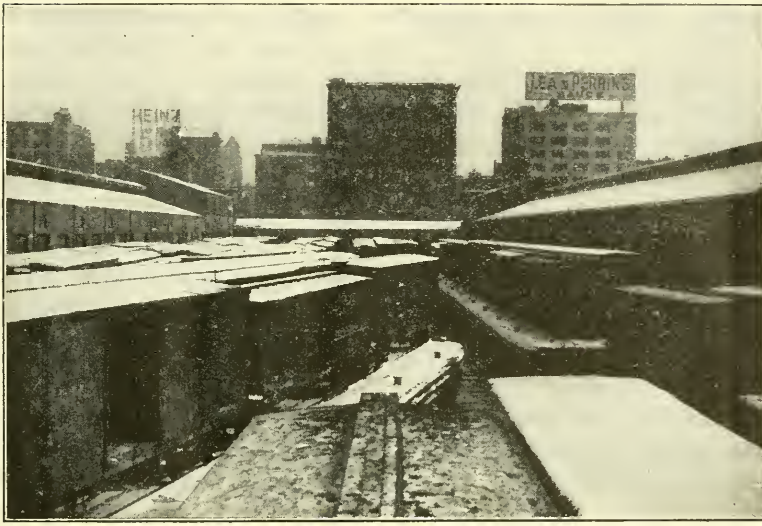


FIG. 1.—RAILROAD CARS ON CAR-FLOATS IN SLIPS IN MANHATTAN.

FIG. 2.—FREIGHT CONGESTION IN A NORTH RIVER RAILROAD PIER.

the fairway of the North River, particularly in the Chelsea section, where the river is narrowest.

There are now nearing completion and under construction a number of other ships of the *Olympic* class, and even larger. Every one of these ships is built for the New York trade, and application has been made to berth them in lower Manhattan. Nearly a year has passed since the temporary extension was granted, and, therefore, it is now absolutely necessary to face this situation squarely.

The City owns and has possession of the property on the North River between Gansevoort and Little West 12th Streets, and this is practically the only property on the lower Manhattan water-front which is under its control. It is set aside as a market, handling general supplies, refrigerator goods, etc., and receives nothing from the water-front. Formerly it was situated in front of the present Washington Market, between Fulton and Vesey Streets, but the necessities of commerce compelled its removal, and it was located at its present site, from which it must again be moved. The water-frontage of this market is occupied as an oyster basin, and there are three small piers utilized mostly as open wharfage.

The necessity of using this space between the two finest steamship installations in the harbor as a market for oysters, poultry, and meats, is not apparent. Application has already been made looking toward legislation for the removal of this market, and, when this is accomplished, a basin may be dredged inland creating berths for two ships 1 000 ft. in length. These, while not giving a permanent remedy, would afford immediate relief, and, by keeping them in the control of the City, could probably take care of the larger ships until better arrangements could be made.

In the pierhead line between the Battery and Gansevoort Street there is a bow which, if straightened out, would permit of constructing piers 1 000 ft. long in the vicinity of Canal Street without digging inland (Plate VI). It is believed that the Secretary of War will at least consent to straightening this line, and, if the railroads which now occupy much of this section can be accommodated in any other manner, a very fine series of steamship piers 1 000 ft. long can be created in a locality where it seems most desirable.

Diagonal piers have been considered, but there appear to be many objections to them. At ebb tide a steamer, backing out from its berth, is compelled to turn more than 90° while floating down the river, and, with diagonal piers, under similar tidal conditions, would back into the river practically pointed up stream. Piers are now built at right angles to the river, and considerable space would be lost at the junction between the straight and diagonal piers.

It is suggested that between 23d and 30th Streets long piers can be created by dredging inland, but this could not be done without

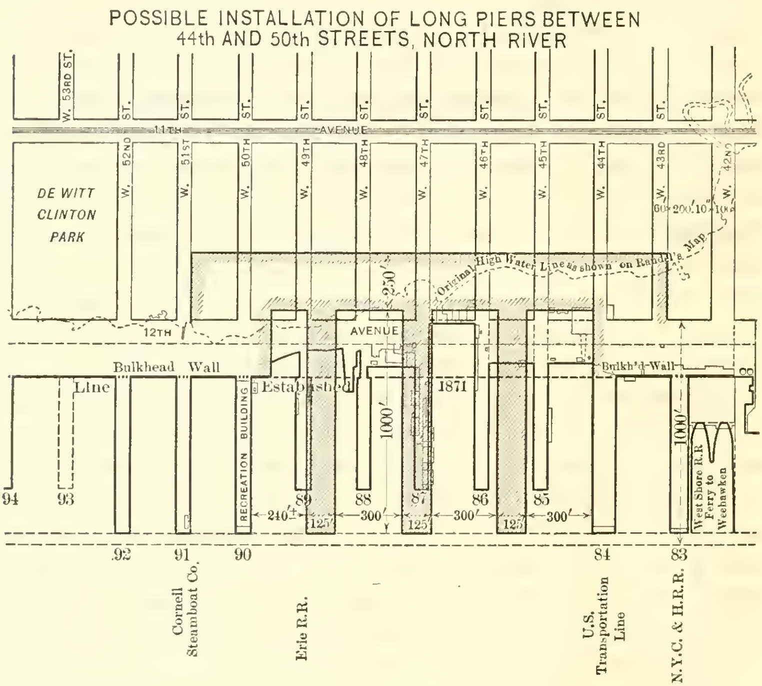


Fig. 3.

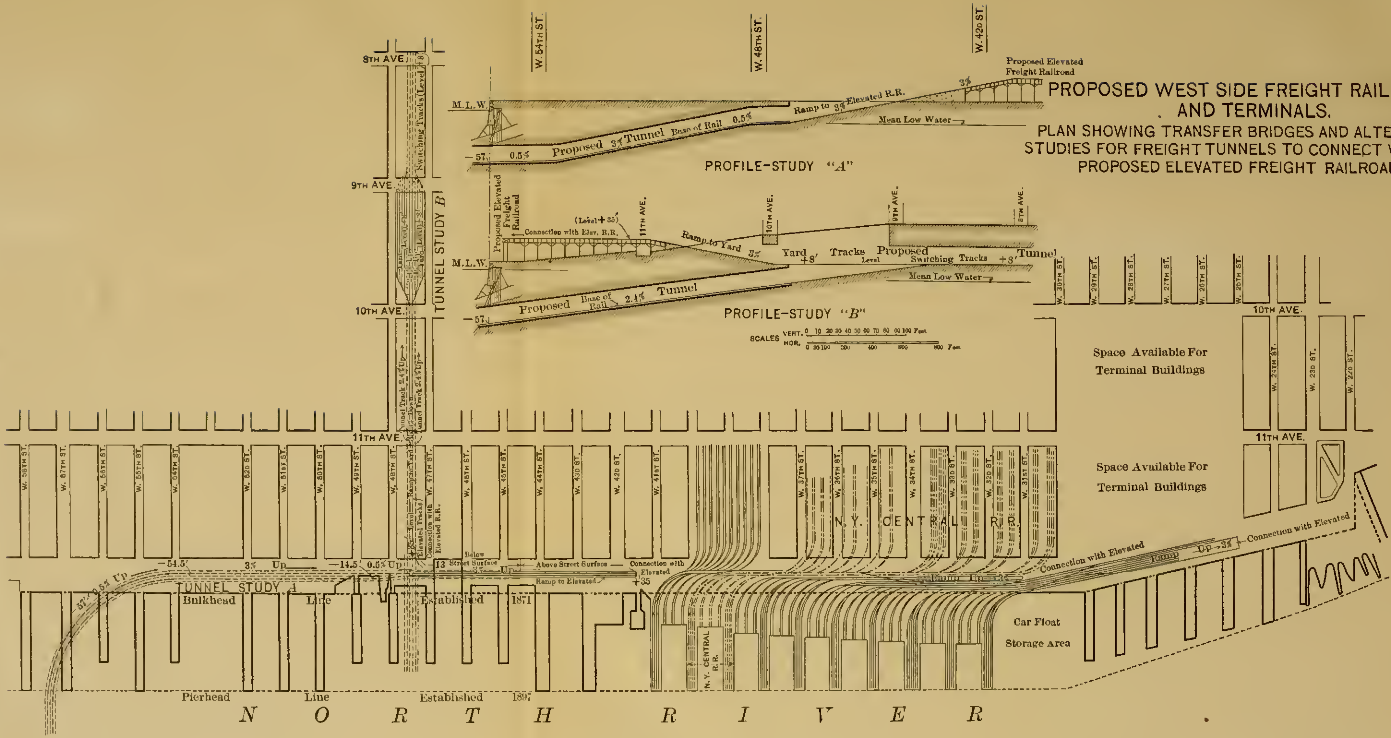
disturbing water-front leaseholds and upland which is valuable for railroad terminal purposes in this district, which would be very undesirable.

Fig. 3 is a study of a scheme for long piers between 44th and 50th Streets. In this locality, the City owns the street ends, but otherwise all the bulkheads are privately owned; and these bulkhead rights would have to be acquired, as well as the upland, in order to make such an improvement.

PLATE VII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXV, No. 1222.
CRESSON ON
WATER-FRONT FACILITIES,
NORTH RIVER, NEW YORK CITY.

PROPOSED WEST SIDE FREIGHT RAILROAD AND TERMINALS.

PLAN SHOWING TRANSFER BRIDGES AND ALTERNATIVE STUDIES FOR FREIGHT TUNNELS TO CONNECT WITH THE PROPOSED ELEVATED FREIGHT RAILROAD.



In accordance with a terminal plan for the future, as well as the present, the Department of Docks has recommended that long piers for steamships of the first class should be progressively provided to the south of the Chelsea district, instead of to the north. This will leave river navigation freer and will better permit of railroad terminal extension in the future. If temporary needs should be considered the controlling factor, however, the district between 44th and 50th Streets, which is not now required for railroad terminal purposes, might be utilized.

It should not be the policy of the City to build expensive bulkhead walls and façades to any piers which are now to be constructed for long steamships; but it should be made as easy and inexpensive as possible to extend slips inland, should the coming of even larger ships require this. The Chelsea piers, with their expensive bulkhead walls and façades, were scarcely finished when they were found to be too short to accommodate the ships which the lessees of these piers had under construction.

It is impossible to foresee the limiting length of ocean steamships, though it appears to be certain that there must be a limit. It seems unlikely that they will be built of much greater draft than at present, inasmuch as the channel depth at practically all harbors is sufficient only for the present draft. It is possible, however, to add to the tonnage of ships by increasing their beam without increasing their draft, and it does not seem impossible to consider that there will be ships even longer than those now constructed. The limiting factor appears to be an economic rather than a physical one. Large ships are very expensive to build and to run. If they could be utilized to their maximum capacity at all times, they would no doubt be very profitable, but it seems unlikely that they will run at maximum capacity, except in certain seasons, and the matter of economy, it is believed, will determine the limiting sizes. The use of the finer ships for cruising purposes in the off season probably could not include these larger vessels.

Railroad Freight Terminals.

Explanation is necessary as to the railroad occupation in lower Manhattan. Railroad cars are placed on car-floats in New Jersey and in the early morning are towed to the Manhattan water-front, where they occupy the slips between piers. Freight from these cars is discharged on the piers, from which it is carted on trucks; the cars are then reloaded with freight delivered by trucks at the bulkheads. This means that a floating railroad yard passes from New Jersey to Manhattan every day, remains in Manhattan during the day, and is floated back to New Jersey in the evening. There are usually from 1 500 to 2 000 freight cars daily standing in the slips in lower Manhattan.

The freight which is handled in this manner consists mainly of food supplies and raw materials for fabrication and consumption in Manhattan itself, and, on account of the inadequate space which the railroads now have, as much freight as possible is diverted from Manhattan. Figs. 1, 2, 4, and 5 show the method of railroad occupation, and the congestion of freight and trucks incident thereto. The existing condition, therefore, is that, with the water-front very much congested, there is a continued demand for more facilities by the railroads, and under the present methods, this could only be supplied by the further exclusion of steamships. The freight business which the railroads do with the steamships is handled mostly by lighters from New Jersey directly to the ships alongside, or to the steamship piers.

The piers which the railroads now occupy were not for the most part designed for railroad use, and it is no doubt possible that others could be designed which would have a greater capacity for handling freight. It does not seem that this would solve the problem in the North River, because, if the capacity of the pier is increased, the congestion of trucks along the marginal way would be correspondingly increased, and this congestion is one of the principal factors in the problem.

In practically all European ports, the water-front is under more complete control by the municipality than in New York, and mechanical devices for freight handling are everywhere in evidence. In New York, however, there is very little in the way of mechanical appliances, particularly for handling package freight. It would be difficult to design a system of carriage which could handle this freight quickly and cheaply, coming as it does in all shapes and sizes. An earnest effort is now being made by the Department of Docks to find some system which will decrease the expensive hand trucking now necessary. European freight cars differ from those in America, as they are

FIG. 4.—CONGESTION OF TRUCKS ON NORTH RIVER MARGINAL WAY.

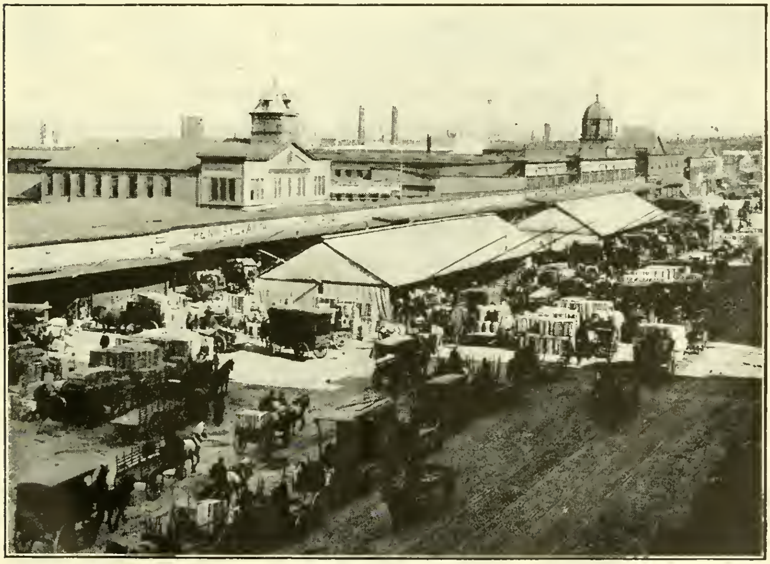


FIG. 5.—CONGESTION OF TRUCKS AND FREIGHT ON THE NORTH RIVER MARGINAL WAY.

smaller in size and capacity, and generally have the top open—canvas covers being used extensively. It is much easier to devise means for placing freight in an open car than in an American box car.

At the Port of New York there are several private companies which are admirably organized and operated, and the fact that they, with complete control over their water-front, factories, warehouses, and connecting railroads, have not applied mechanical devices more extensively seems to indicate the difficulty of handling package freight except by hand, horse, or motor truck.

Surface Tracks of the New York Central Railroad.

The New York Central Railroad has the only all-rail connection to lower Manhattan. The line follows down the westerly water-front from Spuyten Duyvil as far as 60th Street, and reaches a terminal at St. John's Park, bounded by Varick, Hudson, Laight, and Beach Streets, by surface tracks on Eleventh Avenue, Tenth Avenue, West Street, and Canal Street.

This all-rail connection, though very inconvenient, dangerous, and expensive, is of great value to the City, and, by reason of its existence, a strong competitive influence is exerted over the other rail carriers. There has been agitation for cutting these tracks at 60th Street or at 30th Street and requiring the New York Central to carry on its business in lower Manhattan by car-floats in a manner similar to that followed by the other railroads, but it is believed that it would be unfortunate to adopt such a plan, in view of the existing congestion on the water-front and the value to the City of this useful and competitive facility. In addition, it is questionable whether these tracks could be cut without the approval of the New York Central, or without providing a relocation for them.

New York Central's Plans.

Under recent legislation, the New York Central Railroad has submitted plans to the Board of Estimate and Apportionment showing its desired improvement. Above 60th Street, these plans show additional tracks, and types of covering in front of park property. Below 60th Street, they show an elevated railroad, four tracks as far as 30th Street and two tracks as far as Cortlandt Street, with an elevated spur connection through Canal Street to an enlarged St.

John's Park Terminal, necessitating an elevation of the Ninth Avenue Passenger Elevated Railroad over the proposed freight road, and a general electrification of the entire line.

In the district below 60th Street, the plans call for permanent overhead rights to be acquired by the New York Central Railroad, its present surface rights to be surrendered. To grant these exclusive rights to this railroad would give it such an advantage over the other roads now doing business at this port that it would be likely to give it a monopoly to the disadvantage of the other roads and the City's commerce.

The congestion of freight on the piers, bulkheads, and marginal way, and of trucks on the latter is shown by Figs. 1, 2, 4, and 5. The expense to the railroads of maintaining these terminals is great, and, to the merchant, it is usually much greater than the freight rates, because of truckage delays.

General Plan for Reorganization.

Calvin Tomkins, Assoc. Am. Soc. C. E., Dock Commissioner, in his studies of the problem with the writer and the engineers of the Dock Department, believes that another method can be found for carrying on the railroad business, namely, by conducting the joint railroad business over a public elevated railroad operated electrically.

A subway has been considered, but it is not thought that it can be constructed economically or operated satisfactorily on the Manhattan water-front. There is little objection to a subway or tunnel for freight purposes as a connecting link between terminals, but, as a distributing line with sidings leading therefrom, the operation would be very difficult and dangerous.

The New Jersey roads, by transfer bridges such as they use in New Jersey, can make a connection with a municipal elevated railroad in the district between 30th and 40th Streets, and from it can discharge their cars and freight into terminals anywhere along the easterly side of West Street, with opportunity for expansion, and the New York Central can have access to it directly from its yard at 60th Street.

With this railroad built and under the control of the City, and available for use by all the railroads, it is believed that conditions will be created which will enable the City to devote to maritime


MAP
OF A PORTION OF
THE BOROUGH OF MANHATTAN
AND THE ADJACENT
NEW JERSEY SHORE
WITH RAILROAD LINES AND TERMINALS
SCALE IN FEET
1000 0 1000 3000 5000 7000
NEWARK BAY
PASSAIC RIVER
HACKENSACK RIVER
SNAKE HILL
PROPOSED CONNECTING FREIGHT RAILROAD
PROPOSED JOINT R.R. YARD
JERSEY CITY
HOBOKEN
WEEHAWKEN
UNION
NORTH RIVER
MANHATTAN
EAST RIVER
Governors Island
Bedloe's Island
Ellis Island
Blackwells Island
CITY HALL
CENTRAL R.R. OF NEW JERSEY
NEW YORK SUSQUEHANNA & WESTERN R.R.
NORTHERN R.R. OF NEW JERSEY
WEST SHORE R.R.
NEW JERSEY JUNCTION R.R.
PROPOSED TUNNEL
PROPOSED ELEVATED FREIGHT
Morris Canal

commerce a large portion of the water-front which is now given over to railroad car-floats.

Studies have been made for transfer bridges at the lower end of Manhattan connecting with the elevated railroad. This is entirely feasible, but it is thought that the conflict of movement on the elevated railroad in opposite directions during the rush periods will make it difficult to operate. However, connection in lower Manhattan can easily be made.

There is no disputing the fact that the railroads terminating in New Jersey, which are compelled to depend on car-floats at present, will be at a disadvantage with the New York Central, but the disadvantage would be much greater if the New York Central were permitted to monopolize this proposed elevated railroad.

The connection by transfer bridges, whereby the New Jersey roads can have access to the elevated railroad, has been worked out with great care, but is capable of modification in almost any respect. There is nothing complicated about the operation, or which is not now done at many points in the harbor. The difficulties to be overcome appear to be lack of speed, expense of operation, and congestion. The new arrangement will no doubt require more time between the terminals on the easterly side of West Street and the bridges in New Jersey, but it is believed that, by arranging a proper schedule, shipments can be made during a large part of the day, thus getting many cars into New Jersey much earlier than at present.

The cars on car-floats now form a floating yard in Manhattan which must be maintained as a complete yard in order that the runways on these floats may be utilized until the closing time in the afternoon.

As to expense, the railroads are now operating under great congestion and expense in New York in improperly organized terminals. They are subject also to very heavy rentals for piers and bulkheads, and the car-floats which they now use have but two tracks and are of small capacity. With proper organization, the elimination of expensive pier rentals, and the provision of proper floating units, it is believed that they could afford to pay for service over the distributing elevated railroad.

As to congestion at the transfer bridges, it should be remembered that at present, in a single slip between two piers, as many as twelve car-floats are placed every morning and removed every evening, and there does not appear to be any great difficulty or congestion in this movement. If all the railroad business cannot be done by car-float transfer bridges between 30th and 40th Streets, at least there is capacity for a large part of it. The necessity for expensive yards in New York for assorting cars does not exist, as the trains which are destined for certain terminals in Manhattan now occupying the water-front, but which then would be located on the easterly side of the marginal way and be served by the elevated freight railroad, can be made up in New Jersey, as at present.

The plan in general for car-float transfer to the elevated railroad has not been accepted by any of the railroads, and this is perhaps natural in view of the disinclination to operate jointly, and the fear that any admissions may be used as an argument for forcing them from the water-front to the elevated railroad. The railroads probably do not think it desirable that the water-front which they now occupy should be surrendered to coastwise steamships which, as previously stated, are connecting links of other railroads terminating at southern ports, and competitors of the railroads coming to Jersey City. This business, the New Jersey railroads suggest, should be diverted to other parts of the harbor; but, considering all things, it is safe to believe that the New Jersey railroads will oppose granting to the New York Central permanent exclusive franchises and rights on the marginal way.

The foregoing plan for connection from the New Jersey railroads to the proposed elevated railroad by car-floats and transfer bridges is not the best permanent solution of the difficulty.

Freight Tunnels.

It has been proved that tunnels for full-sized rolling stock can be built successfully under the North River. The final solution of this problem would include an all-rail connection from a joint assembly and classification yard in New Jersey to the elevated railroad in Manhattan, thus entirely doing away with the necessity for car transfers by car-floats. With this tunnel connection, a very much more economical system of freight distribution in Manhattan can be had with the use of a very small water-front.

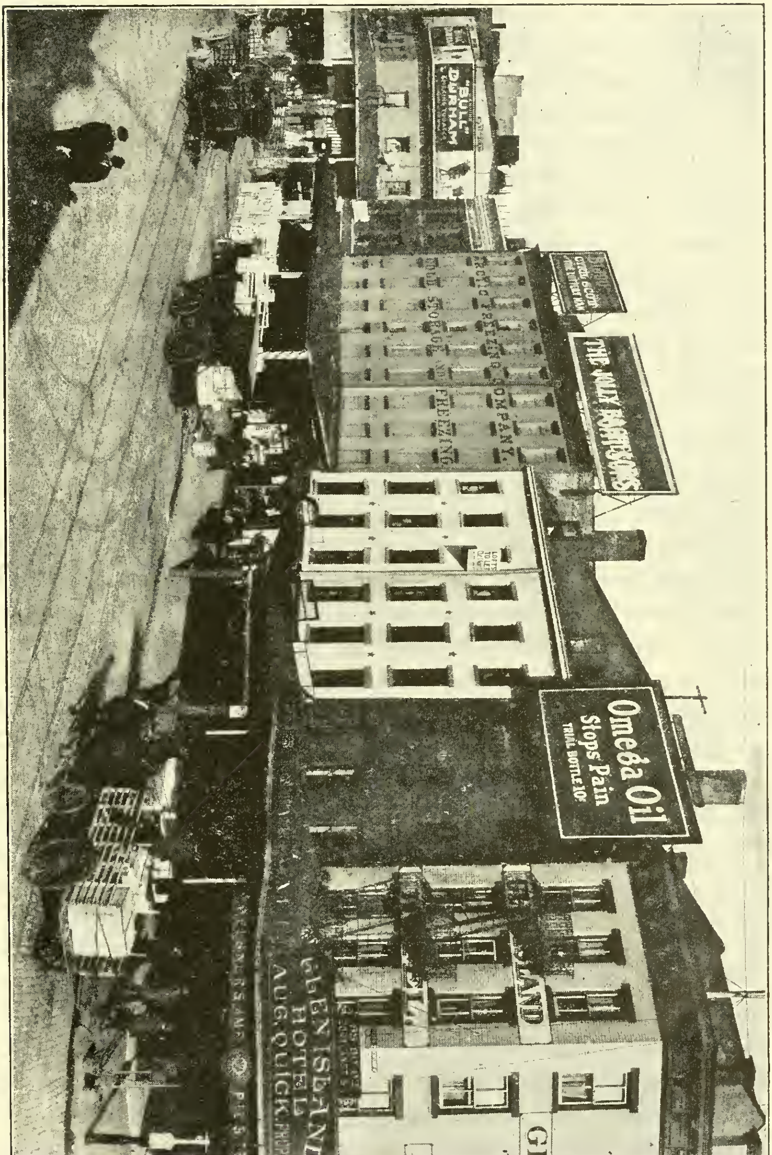


FIG. 6.—CHARACTER OF MAJORITY OF BUILDINGS ON EASTERLY SIDE OF WEST STREET, MANHATAN.

Plate VII shows a study for an all-rail connection between New York and New Jersey from the general classification and assembly yard in New Jersey, with spurs leading to all the New Jersey roads, so that all would have access to the yard. From this yard, by tunnel, a connection is shown to the proposed elevated freight railroad.

The small plan on Plate VI shows four existing railroad yards in Manhattan between 25th and 39th Streets. Considerable business is done in these yards with the use of a very small extent of water-front, and it will be possible immediately to make a connection to the elevated railroad from these yards, as indicated on this plan, so that, in the early stages of the undertaking, the railroad will be available, not only for the New York Central, but also for the Baltimore and Ohio, the Lehigh Valley, the Erie, and the Pennsylvania Railroads.

Plate VIII shows a suggested arrangement of transfer bridges, ramps, and tunnel connections, laid out so that the New York Central may not be prohibited from properly using its proposed terminal north of 30th Street. This general plan would have the effect of giving the City an adequate freight service by the New York Central, and would cause the removal of the tracks from the surface below 59th Street. It would develop terminals on the practically unused property on the easterly side of West Street. It would create a demand for service by the other roads, and, in supplying this demand, would gradually move the railroad cars from the water-front and afford opportunity for expansion on property now used as indicated by Fig. 6, which property would be available on the lower floors for freight terminals and on the upper floors for warehouses, factories, and other commercial uses.

The method of construction of the proposed elevated railroad, the method of operation and control, and other details, can be worked out with the assistance and co-operation of the railroads.

By creating better facilities than now exist at the center of the island, there would be a tendency to draw from that section a large part of the business carried on in a very congested way and subject to great expense for trucking; and, by providing another method in which the railroads can carry on their business, it will make available the lower Manhattan water-front for steamship purposes.

In almost every other harbor, the railroads carry on their business with the City in a district away from the water-front, and that principle should be followed here under the plans proposed. One of the most important features of these plans is that it can be carried out without disturbing existing conditions. It should not be the plan to draw manufacturing to Manhattan or to develop Manhattan at the expense of other parts of the port. Studies have been made and reports published showing plans for the development of practically all parts of the harbor.

Under a recent constitutional amendment, dock bonds which are invested in self-sustaining enterprises are eliminated from computation in fixing the debt limit of the City, and are available for further investment. The dock bonds which will become available in this manner, it is estimated, will amount to $70 000 000, and it should be the City's policy to invest this fund so that it will immediately or very soon earn its interest and sinking fund charges and in turn be removed from the debt limit, thus giving the City a working capital for dock improvements.

The plan of the elevated railroad is not a new one, and many others have been considered, but that outlined herein appears to meet nearly all the conditions. Criticism of the general principles of this plan should be constructive in order to be of any value. The difficulties of operation and organization are recognized, but this plan provides a solution, at least for some of them.

These plans have been worked out at the instance of Dock Commissioner Tomkins, and are the result of long study and intimate acquaintance with the features of the problem. In making the studies, details, and general plans, the Dock Department has consulted with various interests in the port, in order to obtain criticism from every source, and has had the aid and co-operation of William J. Barney, Jun. Am. Soc. C. E., Second Deputy Commissioner, Charles W. Staniford, Chief Engineer, S. W. Hoag, Jr., Deputy Chief Engineer, and R. T. Betts, Assistant Engineer, all Members of this Society, and the general staff of the Department.

DISCUSSION

S. W. Hoag, Jr., M. Am. Soc. C. E.—Mr. Cresson has struck the key-note of the present situation affecting the future of the Port of New York when he refers to conditions and the formulation of a policy which demand the re-organization of the port. The speaker is disposed to lay some stress on this expression, because it emphasizes the fact that the further progressive and up-to-date development of the water-front has brought the City face to face with the necessity for wise discrimination between the past and future ways and means. In view of the agitation which this subject is receiving, and the efforts being made to arouse public sentiment in regard to the situation, it is necessary, in arriving at an intelligent conclusion, to understand and to appreciate at its full value all that has been done by the City of New York leading up to the present time. Mr. Hoag.

Particular attention is called to this feature because of the fortuitous circumstances which have led to the present situation. In 1870, when the systematic development of the City's water-front began, no one but an infallible prophet could have foreseen, either the abnormal growth of the Metropolitan district, or the momentous improvements, in the matter of inland water communication with the Great Lakes, as exemplified by the New York State Barge Canal, and the great inter-ocean communication, as exemplified by the Panama Canal, each of which, the one interstate and the other international, has a direct bearing on any consideration of adequate facilities for the Port of New York.

When, by an Act of the Legislature, the Department of Docks was organized, in 1870, nearly half a century ago, it should be noted that the authorities, at the inception of their work, advertised for all persons who were sufficiently interested in the improvement of the water-front, or who had any ideas on this subject, to attend hearings, discuss schemes for improvement, and offer suggestions. Numerous propositions were submitted, and are now on file in the Department of Docks and Ferries. Surely that was the opportunity for any prophetic announcements or estimates.

The resulting "New Plan," submitted to the Board of Docks by General McClellan, then Engineer-in-Chief, was so far superior to anything proposed by those who responded to this invitation, as to place most of the suggestions in the category of curios. At any rate, the conditions existing at that time, and for many years after, in the Port of New York, and it might be said in the marine world, affecting the City of New York, which was Manhattan Island with the addition four years later of that portion of the Bronx west of the Bronx River, did not justify anything more elaborate than, or particularly different from, what was then designed. The New Plan had the approval of all

Mr. Hoag. commercial bodies, and for years has been the primal force in making the Port what it is to-day. Relative to this matter, it should be borne in mind that the water-front improvements made by the City have been self-sustaining, the annual net revenue received from the operation of all the properties under the jurisdiction of the Department of Docks and Ferries, as of December 31st, 1910, being $3 000 000.

The conditions that have arisen in recent years were not foreseen, nor could they have been foreseen; for, while the importance and supremacy of the Port were inevitable from the start, the magnitude of its growth, within the period between 1870 and the present day, was not even dreamed of. Take for illustration the increase in length of transatlantic steamers as indicated by the vessels of the Cunard Line. In 1840 its longest vessel was between 200 and 225 ft. In 1870 the *Russia* was 315 ft. long, an increase of 175 ft. in 30 years. It was reasonable to expect a corresponding increase, at least, during the next 30 years, but in 1900 the *Campania* and *Lucania* were 625 ft. long, or only 75 ft. more than what might be termed a normal increase. In the next nine years, the longest vessel entering this Port, was the *Lusitania,* of the same line, with 790 ft. The jump in 39 years was from 375 to 790 ft., or 415 ft., the increase being greater than the length of the longest steamer in 1870. Two years later the *Olympic,* of the White Star Line, the largest steamship in the world, with a length of 862 ft., entered the harbor, and has been docked successfully at regular intervals at the Chelsea Section ever since. This is only one instance, cited to show the rapidly increasing magnitude of port conditions covering the latter part of the period during which the Department of Docks and Ferries has managed the City's water-front.

The combination of these almost anomalous strides in magnitude, involving both marine and railroad requirements, is what has created the condition which to-day demands thoughtful consideration and immediate relief. No port in the world has had the experience of New York in these questions, and when the development of the water-front was originally planned, there was no port from which suggestions could have been obtained leading to the adoption of plans which would have avoided the present difficulties. Most of the foreign or European ports are artificial, and more or less confined to comparatively small areas, and in no instance is there the God-given opportunity for expansion that exists here. The unique experience of New York during the past decade, and the present conditions, must furnish the guide for other ports in the future, and some of these have already begun to profit by the lesson, and have inaugurated extensive improvements in their facilities.

When the symptoms of this unusual activity first became manifest, about 12 years ago, there was an available stretch on the North River water-front of Manhattan which was conveniently located and well adapted for the necessary expansion. This frontage was utilized by the development and construction of the Chelsea Section Steamship Terminal, which has been built, occupied, and operated, within the last ten years, by the largest steamships that enter any port in the world. Any further expansion or utilization of the North River Manhattan water-front for such purposes must depend on the reorganization of the lower Manhattan water-front by the elimination of the "marine car yards," of the various railroads, which occupy about 47% of this section. Mr. Hoag.

There are three terms which the experience of New York must inevitably conserve in any modern comprehensive port development—articulation, co-ordination, and organization. These terms speak for themselves; for, without articulation, a heterogeneous mass of units, having no connection with each other, is bound to result. Even with articulation, but without co-ordination, confusion is apt to follow; and with articulation and co-ordination, but without organization, there are bound to be periods of spasmodic confusion and lack of cohesion. It is the recognition and observation of these three factors that make this subject so important, and its discussion so timely.

The loyal spirit of the New Yorker for his native city, and his inextinguishable pride in the Metropolis, are bound to make themselves manifest in no uncertain terms whenever the occasion arises that demands the consideration of anything that appears to be necessary for the maintenance of its supremacy.

The City of New York moves on lines which, to a stranger, might appear to be inconsiderately slow in many things. The spirit of the metropolis in such matters is extremely conservative, and is opposed to experimenting with the people's money. They had the electric light in the streets of Duluth before New York City ever adopted it; Chicago had its Masonic Temple before the sky-scraper appeared in New York; the trolley car was in operation all over the United States (even in Brooklyn, prior to consolidation) before it appeared in Manhattan, but when it did come, it was the underground trolley. In other words, the motive spirit of New York City is to prove the successful accomplishment of many so-called improvements elsewhere, or to "let the other fellow try it out," before adopting it.

All of this is pertinent to the present attitude of the Department of Docks in its desire and attempt to obtain the very best solution of the North River water-front problem with its attending environment, for it should be recollected that this proposed solution is the result of a study of conditions in all the big ports of Europe and the United States; but, in so far as Manhattan is concerned, there is probably no place in the world that is confronted with such adverse geographical and local conditions in meeting the requirements for all-rail connection with the rest of the United States without an extensive outlay;

Mr. Hoag. and the logical solution to-day would doubtless have been considered extravagant and unnecessary even a quarter of a century ago.

Mr. Harding. H. McL. Harding, Esq.*—Mr. Cresson has stated so fully the conditions in reference to the terminal arrangements for the transference of miscellaneous freight along the North River water-front, that little can be added.

From a conservative engineering standpoint, it seems as though the plan outlined should receive the commendation and support of all civil engineers, and especially of those who are familiar with the energetic and untiring exposition of Calvin Tomkins, Assoc. Am. Soc. C. E., Commissioner of Docks and Ferries, for the furtherance of these greatly needed improvements so essential to the foreign and domestic commerce of the City of New York.

This complete plan may be considered as the best suggested after twenty-four years of commissions, reports, and discussions. The necessity for improvement has long been recognized.

There are two general facts which cannot be gainsaid, and concerning which there should not be any discussion. One is that there are a number of steamship companies (twenty, the speaker believes), clamoring for berthing facilities to land and receive cargoes of industrial freight. The other is that better facilities for freight receipts and deliveries are desired, not only by the shippers and consignees, but by the railroad companies.

Clearing away the mass of objections, some sincere, some not disinterested, and some due to a proneness to argument, there seems to be a consensus of opinion as to the methods in general, and any discussion is rather on the details of application and operation.

There are several questions asked: One is, "Is it possible to find room anywhere for these steamship companies along this water-front?"

By considering the railway problem first, the answer to the above may be more clearly understood. Chicago has 250 freight stations, Philadelphia and other large cities also have many, even when there is practically one railway company in control. These stations are located in different sections of a city so as to obviate long drayage hauls by the merchants and also to prevent the impending and increasing package-freight traffic congestion.

That a number of such freight receiving and delivery points are necessary, has long been conceded; but there will not be economical service at these sub-stations unless there are rail connections with some central freight or transfer station. For Philadelphia, one central transfer station is at Mantua where the cars from the sub-stations are assembled and the freight is redistributed so as to obtain greater carloads for designated points. At this same transfer station there is a

* Consulting Engineer, Department of Docks, New York City.

further interchange and reloading of both the in-bound and out-bound local car load freight. Mr. Harding

In New York, below 23d Street, this sub-station collecting service, including the assorting, is carried on at the bulkhead sheds, these being the station, and cars on floats in the slips being the freight yard. There are a few sub-stations away from the water-front, as at St. John's Park which is connected by surface railway tracks. These tracks should be removed from the streets, in the interest of both the city and railway. If the railways did not occupy these bulkheads, piers, and slips, they would be available for the steamships. There are some twenty-eight piers occupied by six railway corporations, and in some cases four of these piers, within a few blocks of each other, are under one control.

Sub-stations are essential to the business of the merchants of New York, but are there not other places where they can be located more advantageously than at the piers and slips, thus avoiding the present congestion, and yet releasing the slips and piers to water transportation? This congestion is so acute that in a bulkhead frontage of 216 ft., as much as 400 tons of package freight is handled during 3 hours. A freight station should have at least 800 ft. for this service. As shown by Mr. Cresson, there are such places, and, due to the long, narrow configuration of the Island of Manhattan, stations can be distributed so that long hauls are not necessary. In addition, there are modern mechanical methods of freight transference which will greatly increase the capacity for any given space to three and four times that where the floor area and one level are occupied by trucks.

By good fortune, this portion of Manhattan has not yet been improved, but is largely occupied by rookeries and tenements. This space is along the east side of West Street below 23d Street. As it would not be available unless there should be rail connection, and, as surface tracks cannot be permitted, and a subway has been decided as not practical along the lower portion of the Island, an elevated railway is the only alternative. This is the whole story condensed, and is the final conclusion after many years of deliberation.

There are the following pertinent questions:

First.—Will there be the necessary space for the cars, the drays, and for receiving and delivering the freight?

Yes, but not for car placement for assorting according to cities. This should be done elsewhere, at a transfer station, either above 23d Street, where the land east of the marginal way is already owned and occupied by the railroads, or in New Jersey, as shown on Mr. Cresson's map, Plate VIII. Instead of the twenty-eight sub-stations now within this 2½ miles from 23d Street to the Battery, a less number should be sufficient and yet permit the stations of the different railway controlling corporations to be equally accessible to their

Mr. Harding. present or prospective customers. These stations would occupy the first and second stories of high buildings constructed on the east side of West Street. The third floor could be used for stored or held-over freight.

The tonnage to be transferred daily, including the in-bound and out-bound freight, would not exceed, as a maximum, 300 tons per hour per station for some time to come. At one station, now congested, 700 tons are handled daily, and, at another, 1 000 tons; but, within about 3 hours in the afternoon, more than half of the above tonnage is handled. At present, the freight must be removed as quickly as received, to keep the bulkhead frontage clear. This would not be so necessary if there should be more room, though it would be advisable in order to secure quick car loading. The space required for transferring 300 tons per hour will be given and described later, with other requirements.

Second.—Can dray congestion and delay be avoided?

Yes, absolutely; provided the whole first or street-level floor is reserved for the drays and their platforms, and the second floor for cars, car platforms, and the longitudinal openings between the floors, the station being equipped with overhead transferage runways, electric transfers, and transfer trailer hoists.

All platforms and other space within the range of the machinery should be served directly by this machinery; there should be no re-handling by manual labor, and there should be continuous rapidity, that is, no lost-labor time waiting for the return of the machinery. One machine should follow another so closely that the operation should be practically continuous. The out-going freight should be placed on flatboards when received from drays, each flatboard holding one consignment for convenience in weighing, and, though the consignments are separated, yet at least three consignments should be hoisted and conveyed simultaneously, each being weighed when passing over a section of the track connected with scales.

Each string of cars should be loaded to full capacity, and, as soon as ten or more railway cars are fully loaded, they should be despatched to the transfer station. It should be possible to load any car from any platform, the load passing by gravity rollers within the car, after it has been lowered by the trailer hoist. Similarly, the in-bound freight should be taken from the cars, but held awaiting the arrival of the drays of the consignees. There is a space reserved on this second story for storing some 2 000 tons or more of in-bound freight.

Within a space of 200 by 200 ft. on the ground floor there could be transferred more than 800 tons per hour from drays. This is an average load of 1 ton per dray, due allowance being made for the dray area and platforms. As 300 tons would be the maximum, the foregoing is given to show the approximate capacity of a frontage of one block. Two stations, therefore, could be easily accommodated within this space, avoiding dray delay and congestion either within or without the station.

On the second floor, 600 tons per hour, and even more, could be loaded into cars brought from the elevated railway on the same level and despatched. There would be space here for the cars, platforms, and openings. In an engineering report, the size of the platforms, the number of drays, the average loads, the space occupied per dray, the time to load or unload, the average tonnage per car, the time required to load and unload the cars, the space for car platforms and openings, and other data, will demonstrate the correctness of the foregoing statements, but the operation can only succeed by the use of mechanical appliances. This use of machinery has been considered in calculating the area capacities. There would also be ample capacity on the four tracks of the elevated railway for all the car movements, continuing these to such a proportion of the 24 hours as the volume of freight would warrant. Mr. Harding.

Third.—Will the railways now using the bulkheads and slips accede to the change?

This question can also be answered in the affirmative, provided the City will afford such facilities that the proposed locations will remove the present congestion, offer greater and permanent advantages as to economy and rapidity than they have at present, and furnish separate or combined terminals, according to the option of the individual railway companies. The railways, however, want no particular railways to have superior advantages.

It now costs about 40 cents to handle one ton of local car load package freight from the bulkhead frontage into the cars, including the usual operations of inspecting, weighing, routeing, and checking, and 40 cents more to transport the freight over the river, and more than 10 cents additional on the other side for the car movements; probably a total of 95 cents per ton would be a fair average for comparing costs of identical operations.

If the City will provide the elevated railroad, the tunnels as shown on Plate VIII, and the collecting and delivery terminals mechanically equipped, for 4% interest on the cost, on the same basis as the terminals of the water transportation companies (these terminals being self-supporting), and with a fair charge for switching the cars through the tunnel, the railways should be able to give better service and at less cost than at present. This statement as to less cost may be disputed, but if mechanical appliances are properly installed, with a full knowledge of the operating conditions, and according to engineering experience as to what the machinery can do, the greater rapidity and economy can be secured. The advantage of being able to assort into the cars on the floats is largely, if not wholly, nullified by the extra expense of the long trucking, averaging some 700 ft., and the cost of transporting across the river so many cars only partly loaded, together with the expense of the transfer bridge and the subsequent

Mr. Harding. car switching. The following figures may serve as a basis for comparison:

Present Partial Costs, per Ton.

Pier rentals	8 cents.
Expense of handling	40 "
Transporting expenses (actual)	40 "
Switching	7 "
Total	95 cents.

Costs Under Improved Conditions, per Ton.

Station rentals with better and permanent locations (based on greater area and tonnage, out-bound and in-bound, and trucking space), and using only two stories	5 cents.
Expense of transferring from dray to car and *vice versa*	12 "
Transporting to New Jersey and switching	15 "
Mechanical assorting at transfer station in New Jersey	12 "
Total	44 cents,

a total saving of 51 cents per ton.

If these figures are accepted as only approximately correct, where is the claimed advantage of assorting the light loads into many cars on floats, especially as these cars often go to transfer stations?

There are many other conditions which must be carefully weighed, but these brief statements may open the way for a further discussion from which exact figures may be obtained to substantiate the foregoing conclusions.

The railway companies will soon need more room to handle their increasing package freight, and the dray congestion of the merchants has reached the limit of sufferance. Every conceivable plan has been advanced; but, the one cited by Mr. Cresson, of an entire rail haul, from a number of independent sub-stations, appears to be to the advantage of the whole city, shippers and consignees of freight as well as transportation companies.

Mr. Forgie. JAMES FORGIE, M. AM. SOC. C. E.—Mr. Cresson's paper has been endorsed so thoroughly that it seems to be unnecessary to add to that endorsement, but it may be further emphasized. This problem is to restore the water-front to its proper use, to do away with floating railroad yards, and put them in their proper place, and to remove the railroad tracks now occupying the surface of exceedingly busy streets.

Mr. Thomson has referred to the enlargement of Manhattan Island by connecting Governor's Island to it with sea-walls and filling in.

Is it not rather a pity to introduce such a far-in-the-future solution? It would seem that the introduction of such a subject is departing from the issue. The immediate issue is the relief of these difficulties, and the solution is applicable now, and would be applicable to any extension or prolongation of Manhattan. Mr. Thomson states that the extension of Manhattan "could be" done in 5 years. Mr. Cresson's plan can be carried out, including the lines and yard in New Jersey, the tunnels under the Hudson River, and the elevated line in New York City, in 2 years. Mr. Forgie.

Is it not to be regretted that Mr. Cresson introduces a compromise into his paper? Doubtless, however, he has reasons for this. Why should transfer bridges be constructed in the neighborhood of 40th Street, when the tunnel scheme can be completed in so short a time? There are two other schemes in connection with this problem. One of these includes small marginal tunnels, but this involves relief by breaking bulk, while the author's solution brings in the traffic directly on the original cars down the water-front, independent of the street traffic, so that they may be flexibly diverted at the high elevation either to the piers, the warehouses, or the yards on the east side of West Street, as the case may be. The other scheme—and a bad one—is a concentration of transfer bridges at certain places on the front, retaining carriage by float across the busiest harbor in the world, and taking the trains across to the east side of West Street in what seems to be a complicated and impracticable way.

Mr. Cresson deals with an attempt to dissolve serious interferences in the economic handling of freight to and from Manhattan Island, the misuse of the harbor front, and the chaos prevailing in West Street, owing to railroad trains at grade and other traffic which can be handled better elsewhere. In the speaker's opinion, Mr. Tomkins and Mr. Cresson are to be congratulated on having formulated a simple and direct means of accomplishing this at a comparatively small expense, which, it seems, can be more than carried by the saving in the handling of freight alone. Manhattan Island is becoming more and more of a financial center and less of a manufacturing center, and such freight handling as is necessary should be transferred from the water-front as much as possible to areas more capable of development, and fed directly by rail and not by car float. The expense of the car-float arrangement has been pointed out time and again, and, while it cannot be abolished in a day, the policy should be to eliminate this traffic gradually, within economical limits, and obtain direct entrance to the water-front by rail; in this instance, there is a possibility of doing so with benefit to all concerned.

New York has enormous natural advantages as a port, and by a tour around its great frontage, any intelligent observer can readily see a lack of commensurate development and that it is also beyond the

Mr. Forgie. stage of having freight yards in its public streets and on its important water-fronts.

From one's office windows can be seen the conditions pertaining to the greatest transoceanic port in the world, which are more suited to the surroundings of Newtown Creek. Can one doubt that while Manhattan is the great port it is, it would have been still greater as a port for foreign and coasting vessels had the harbor front been available for such a purpose? At the present time the dockage in Hoboken proves this. As it is, the Manhattan front is to a great extent a floating railroad yard, and West Street is in many respects more a warehouse and storage-room for freight and vehicles than a vehicle of dispersion.

Recently the speaker listened to Mr. Bush, of the Bush Terminal Company, who is doubtless an authority on water-front development, and the gist of his statement was to the effect that shipping should not be waiting for port facilities, but that port facilities should be waiting for the shipping, and that the more the shipping capacity of the water-front around New York was increased, the more the hinterland improved and the more the business of the port developed. This paper is a very complete suggestion or plan of what is considered the best means of utilizing this frontage to its capacity by the removal from it of that business which can be handled better and more economically elsewhere, giving greater facilities for the disposal and acquisition of railroad freight, and elevating the railroad tracks which are now a nuisance in the streets.

The freight business in New York is so great, that to sort freight cars on floats while trucks wait outside in the front street is, in this location, surely out of date when it is known that this particular traffic can be accommodated by some other means to the greater advantage of all railroads, and with great economy and saving in time. The plan to gather the freight cars of the railroads in a yard in New Jersey, sort the trains, and bring them into New York without breaking bulk, by means of a belt line in New Jersey, under the North River, and down the water-front on an elevated line, is surely far the best solution of the difficulty.

As far as the construction of tunnels under Bergen Hill and the Hudson River is concerned, that is a simple matter; as far as the elevated railroad is concerned, that is still simpler. There can be no esthetic reasons for prohibiting such a structure on the west water-front.

As a rule the speaker advocates the depression of tracks in cities; but on the harbor front there is no reason for doing this with these freight tracks. On an elevated structure they would be at a suitable level for rapid disposal, either into warehouses or on the piers, whence freight can be handled much more readily than on the ground level.

This scheme would put every railroad on an equal footing; it would bring all freight in from the West, or from anywhere else on the west bank of the Hudson—or for that matter, from the North—without breaking bulk, and would be much more flexible than a tunnel system. Mr. Forgie.

Suppose the tracks were depressed in West Street, or on the water-front marginal street all around the island, one can readily understand that extension into warehouses, freight yards, piers, sidings, etc., from such a subway would in all instances be very expensive, and would waste much useful ground space in approaches and shunting areas; on the other hand, from an elevated line, all such connections would be extremely flexible and comparatively inexpensive. What reason can be given, commercially or esthetically, against an elevated structure with elevated connections on the water-front streets, where enormous quantities of freight have to be handled and where people do not reside?

The only objection to this scheme is the fact that it will add to the abnormally advantageous position which New York now occupies—how much better will the citizens of New York be 30 years hence, when its population will be double what it now is as a result of its accommodating its passenger- and freight-handling business in a scientific manner? If we must increase commercially, we must do as any up-to-date factory does—scrap the old for better means of economical results.

While observing the neglect and lack of appreciation of the extraordinary natural advantages of New York as a port, the speaker cannot but contrast it with the comparatively small ports on the east coast of Scotland, where harbors have been dug out of the solid rock at a vastly greater expenditure per capita than New York would have to meet, and for incomparably less return.

Edlow W. Harrison, M. Am. Soc. C. E. (by letter).—Mr. Cresson's paper should be welcomed as a timely contribution on a subject which, in the minds of many thinking men, is becoming of supreme importance. Mr. Harrison.

All signs indicate that this century is to see an enormous development of world trade, and that the United States, with its unequalled resources and millions of intelligent workers, will lead in this movement, and that the measure of its lead will depend, very largely, on the value of the facilities given by the Port of New York.

The subject and the problem are national, not local, and must be treated on the broad lines of national continental policy.

The writer does not wish to tread on any one's toes, or to belittle the earnest efforts of many honored citizens and distinguished engineers who, from time to time, have interested themselves in the development of the Port of New York, but he cannot help saying that the

Mr. Harrison. interest, in the past, has been largely of a narrow, provincial, and selfish character.

By its natural position and the course of growth of the continent behind it, the Port of New York has become the principal gateway of the nation. All roads lead to it, both from the interior, and across the ocean. There are no indications that its supremacy will decline for many years to come, but, on the methods of its development, and the economy of its operation, as a world port and as a factor in securing the leadership in the trade of the world to the people of this country, the relative welfare of many millions depends.

There has been too much endeavor to concentrate and congest commerce on the Island of Manhattan. The feeling has been: "Our Bay," "Our River," "Our Commerce," "We must hold it, not as a trust for the nation as guardian of its gate, but because we need, and must have, the incidental return, the tariff, or the 'rake off,' from having its commerce touch Manhattan." The attitude has been that of the commercial ports in the sixteenth century; to force the stream by laws and regulations.

To paraphrase the old quotation, we should do as the London of to-day does: "Let us negotiate the bills of exchange and furnish the cash for the commerce of the world. We care not what stevedores handle the freight; what railroad or vessel carries it; or through what port it passes, provided the charges are the lowest obtainable."

London may have carried this principle too far, and allowed her port facilities to fall behind the times, but she is now remedying this neglect.

The writer is glad to say that the present Commissioner of Docks and Ferries has taken a broad view of the necessities for the future of the Port in his capacity as a member of the Joint Commission, appointed by the Governors of New York and New Jersey, to examine into and report on the development of harbor facilities, and it is with the heartiest appreciation of Mr. Tomkins' sincerity that he ventures to criticize certain of his plans, mainly because they seem to him to have been devised without a proper consideration for the actual conditions surrounding the problem, and to be a continuation of the narrow medieval policy.

The Port of New York, taken as a whole, may be said to cover all the enclosed waters inside of Sandy Hook and Throg's Neck, including the Amboys, Arthur Kill, Newark Bay, and the lower Passaic and Hackensack Rivers; Kill von Kull, the Upper Bay, the lower Hudson, and the East River to the Sound. Within these boundaries, and as a whole, this Port is equal or superior to any commercial harbor in the world, for ease of entrance for vessels even of the deepest draft, moderate tide, comparative freedom from ice, good holding ground for anchors, extensive shore line, advantages for shore front improvements, docks, wharves, and piers, at comparatively moderate cost, easy and cheap dredging, and convenience of communication between all its surrounding shores. It is within 24 hours by passenger time, or 48 hours by fast freight, of a population exceeding that of Great Britain or France, and is the market for every possible production known to civilized man. Mr. Harrison.

All parts of this harbor, however, are not equally valuable for the commerce of to-day. In the years before 1865, the Island of Manhattan, constituting the old City, was the most favorable site for commercial operations in the Port and in the country, and it had then attained, and had held for years, the commercial supremacy it still holds. To-day the Borough of Manhattan is probably the most unfavorable and least economical district of the whole Port for handling and transferring freight, and can only maintain its position by expensive and artificial forcing of the natural course of commodities. It is living on old traditions under radical changes of conditions.

A little study of the course and growth of commerce in the past half century will show that this is no careless statement, but a very serious fact. Before 1865, New York's commerce was by water carriage. Her coal came by canal barges *via* the Delaware and Raritan to Amboy, the Morris Canal over the mountains from Easton, the Delaware and Hudson Canal and Hudson River from Rondout, or by sailing vessels from Philadelphia, with a small amount brought over the single track of the Central Railroad of New Jersey to Elizabethport, and then to New York. Wood and charcoal came by sail up the coast from the Chesapeake. Cereals, flour, quantities of food stuffs, and lumber, came through the Erie, Champlain, and Oswego Canals. Meat was largely of State or Ohio raising, and large droves of cattle, hogs, and sheep were driven, on the hoof, from the Mohawk and Genesee Valley to the Hudson, and then carried by steamboat to the city. New York was a city of wood, brick, and Belleville brown stone, or Vermont marble, with cast iron from the Highlands, Troy, or Paterson furnaces—all by water carriage. Fruit, vegetables, and dairy products came by canal or river boats from up river, New Jersey farms, Long Island, or the Sound. Fish and oysters, of course, came by boats. Philadelphia and Baltimore trade was by boat through the canals, as was that of all tidewater New England. Textiles, when not imported from abroad, came by Sound boat from New England mills. Cotton, corn, sugar, and all Southern products were, of course, by coastwise shipping.

As now, one railroad entered the city from the West, but its freight tonnage and capacity for business, compared with to-day, were almost negligible quantities.

Across the Hudson, New York's rail connection for freight to the West was by the united railroads of New Jersey, with yard capacity for about one hundred cars at Jersey City, and the same at South

Mr. Harrison. Amboy. The Erie had only just completed its tunnel, and was still handling its freight by boat, *via* Piermont. The Reading, Lehigh, Lackawanna, and West Shore had not reached New York Harbor. An ocean-going vessel, from 300 to 350 ft. in length and of 3 000 tons burden, was a big ship.

New York Island, with no part of its surface more than a mile from navigable water, with a population of about 900 000, with plenty of labor, which was being added to weekly by hundreds of European workers of the best class, with good banking facilities, and ample capital and credit, with vacant land at reasonable cost, and with good sanitary conditions, was unequalled by any seaport town of the country as a transhipping point for commodities, combined with advantages as a manufacturing center, and a mart for trade, wholesale and retail.

With the development of the country, all this has changed. The commodities seeking the Port of New York have increased in volume many fold, but the increase has been in rail tonnage. Steel rails, larger train loadings, and more powerful engines, have driven the water-carried goods to the rail. Two freight cars out of a train of forty or fifty will convey a good loading for a canal boat at a fraction of the cost and time of carriage; but, with the exception of one road—the New York Central—all the great carriers from the West and South only reach the west side of the harbor with their rails. The great city, with the exception of the Borough of Richmond, is dependent on a forced arbitrary rule to hold her position.

Under present regulations, all points around the Harbor enjoy an equal tide-water rate and free lighterage within liberal limits, notwithstanding the fact that it costs 60 cents upward per ton more to deliver or receive freight on Manhattan and Long Island than on the New Jersey or Richmond side. This arbitrary rule is practically spread over all the tonnage, so that freight which goes no farther east than the tide-water points in New Jersey, pays its share toward feeding the wharves of New York and Brooklyn.

This charge, at last, rests on the producer, or the ultimate consumer, and, in the competition for world trade, must be reckoned with. In the present agitation and study of rates, it cannot forever be overlooked, and some day the West will find out that the ship in the Port of New York may be reached without crossing the Hudson or New York Bay, but at the end of the track, and this arbitrary rule will have to go. Such artificial obstructions to free commerce cannot last in the world of to-day.

It cannot be denied as an economic proposition that the commodities of the continent, not originating or having an ultimate disposal on Manhattan or Long Island, should not be burdened with the cost of transfer. It also seems to be an economic axiom that a manufacturer, obtaining his raw material from over sea by ship, and from the interior

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by rail, and whose products are sent over sea or to the interior, in competition with the world, will not burden his business with the handicap of transfer across the Hudson or New York Bay, unless he receives some equal compensation by so doing.

The Borough of Richmond and the New Jersey side of the Harbor have undeveloped or partly developed water-front which is capable of doubling the commercial facilities of the Port at a comparatively small expense, and these facilities are at the end of the tracks stretching south and west across the continent, with thousands of acres of comparatively cheap, vacant lands adjoining the water.

New York has been possessed of great natural advantages from its beginning, but it has slept on them. It has heard, so often, that its harbor was the finest on earth, capable of accommodating the navies of the world, that it has not noted that the ships of the navies of the world have grown from 3 000 tons, 350 ft. length, and 20 ft. draft, to more than 20 000 tons, 600 to 900 ft. length, and 30 to 35 ft. draft; and that, of that great expanse of surface in the harbor and the two rivers, hardly more than one-third is available for major ships of the present day, not to mention the future.

New York's commercial facilities have grown like a wild tree—with no direction, no plan for the future—for years a football for politicians, and a medium for the transfer of spoil.

At the same time, the National Government has doled out, for the improvement of the main gateway of the country, a total sum of less than $20 000 000, including the cost of the Hell Gate and Ambrose Channel improvements, which is farcical in comparison with the appropriation for other and unimportant harbors.

The whole of Manhattan has grown in the same way—with no plan, no restrictions. To an outsider it has seemed to be a mad race to get as dense a crowd as possible in one spot. Increased land values have called for more floor space to earn taxes; more floor space called for more people to use it; more people called for more subways; more subways for more bonds, and therefore more land values, and so the circuit goes on.

The natural tendency of the people making up its population is to concentrate—a tendency advantageous to the real estate and building speculator, but deprecated by every expert in city building of to-day, and carrying with it evils which will be felt for years. No attempt has ever been made to prevent congestion. Every city move has been toward temporizing with the evil, or encouraging it. Therefore there are sweat shops in Fifth Avenue; fire problems; sewer problems; water problems; an army, greater than ever gathered on a battle-field, rushed lengthwise through the town, packed in masses, twice a day.

Contrast this with the modern scientific work of building or remodeling, in the light of the twentieth century, in the old medieval cities

Mr. Harrison. abroad, for instance, in Cologne, Antwerp, Berlin, Hamburg, or even London. The motive is to spread out, to distribute the people, to restrict the population per acre, to prevent congestion at any point. Manufacturers are forced to an outer zone, where they can have direct railroad connections. The houses of the working people are sanitary, and allow of decency and self-respect; they are usually detached, with plenty of open space; they are in the suburbs, far from the center of the towns, but their workshops are there too. There is land and building speculation, but the speculator is forced to follow the rigid plan, and must build what and how he is told. The market places are in convenient locations, connected by rail. If a maritime town, the wharves and docks are arranged systematically, to secure the transfer of freight with the least cost.

One disadvantage, which we will always labor under in the problem of terminal freight distribution and transfers between land and water carriage, as noted by C. W. Staniford, M. Am. Soc. C. E., Chief Engineer of the Department of Docks, in his report to the Mayor on European Harbors, is the size and weight of our freight cars. The European freight car is built to carry from 8 to 15 tons, and can be shifted easily by 3 or 4 men. It is transferred from one track to another, or across the yard to tracks at right angles, by turning on a light turn-table. The great majority of the cars are flats, the freight being carried under canvas.

Our usage has been toward heavier cars and loading. We look for economy in the long continental hauls by heavy train loads, and, by so doing, we have reduced the cost per ton-mile to a figure which is startling to a European railroad man; but we cannot shift a car without an engine, or without giving up from 200 to 250 ft. of track for switches and clearances. Foreign cars can be handled on a pier or wharf in limited spaces, unloaded quickly by crane, and shifted away rapidly by man- or horse-power. A string of our cars in the center of a pier shed would be a nuisance, and, in most cases, impracticable, especially if the empties had to wait for power to be shifted or removed.

That this is the opinion of the steamship men seems to be proven by the fact that, at any time in the past, the New York Central tracks could have been turned into the piers on West Street, and the piers in Hoboken could be connected in a few hours to all trunk lines; but the tracks are not wanted.

It is true that most of the railroad terminal piers are equipped with tracks, and the cars are loaded and unloaded to and from lighters and other vessels; but these piers are part of the yard system. There is ample standing room for cars, and drilling engines are at hand at all hours.

A study of the track system required at the head of a railroad pier will illustrate the difficulties of handling cars from four running tracks on West Street in connection with a series of piers at right angles to the running tracks, and spaced, say, 350 ft. apart. Mr. Harrison.

The writer agrees entirely with the recommendation of the Dock Commissioner that the pier line between West Twelfth Street and the Battery should be made a straight line, and further, he cannot see why the pier head at Battery Place should not be extended 200 or 300 ft. into the river.

An inspection of the charts will show that the present actual width between the 6-fathom contours at the mouth of the river is less than the same width at the Chelsea Piers.

To one familiar with the limited and crowded fairways of the great home ports of the European liners, through which they are skilfully handled, it is interesting to hear the argument that a waterway ¾ mile wide cannot be reduced without danger to navigation. Abroad there is a strict rule of the road, and it is enforced on water, as on land. We have regulated traffic on Fifth Avenue and other streets, and more than doubled their usefulness. We must do the same in the Hudson River. The relative importance to-day of longer piers exceeds the advantages of width of fairway. The recommendations of the United States Engineers to improve the channel of the East River, thus encouraging greater traffic in an already crowded waterway of much less width than the Hudson, should premise a favorable outlook for pier extension on the Hudson.

Mr. Cresson's assertion of the necessity of docking the great liners at Manhattan piers, and the alternative suggestion that long piers might be built to accommodate such traffic at Staten Island or South Brooklyn, savors very much of the provincial spirit which the writer has already decried in the attitude of Manhattan toward the port, as a whole.

Two of the largest, most successful, and best patronized fleets of liners in the world have always had their docks at Hoboken, and probably 40% of the transatlantic business by regular liners is handled on the New Jersey shore. There is room for several 1 000-ft. piers, with 35 ft. of water, north of Castle Point, and within easy access to the McAdoo Tunnels.

In the veracious history of Mr. Knickerbocker, it is related that on the occasion of the capture of Manhattan by the English, the inhabitants of Communipaw, by all smoking together, created such a fog that they were lost to view, and escaped discovery by the conquerors, who were deceived with the idea that there was only one side to the River and Bay.

As to the proposal for a joint classification yard on the Hackensack Meadows, with a tunnel to Manhattan, and a distributing elevated freight railroad along the west side to be used by the New Jersey roads,

Mr. Harrison.

as well as the New York Central, the writer believes that, if it were practicable, it is inadvisable, in the light of modern civic study. Such a scheme would result in aggravating the congestion on Manhattan Island, perhaps to the improvement of real estate values, but certainly to the detriment of the economical service of the port. To a railroad man who knows what it is to handle a rush of delayed freight at the throat of a terminal yard, the problem of drilling a sufficient number of cars to pay fixed charges on the investment, through the tunnel, or from the bridges, up the ramps, placing and removing cars from a hundred warehouses and piers, while keeping a running track open, is appalling.

If the City of New York is to stand the added fixed charges by general taxation, the structure may be used, to some extent, when desired by shippers, but the cost is certain to exceed the present cost by car-floats.

The writer cannot see wherein the saving is between hauling from the car float piers as now, or hauling from a warehouse floor 20 ft. in the air, for distribution about the city.

A study of the yard room necessary for the comparatively small volume of freight handled by the private enterprise at the Bush Terminal seems to condemn the practicability of this plan.

The idea of a general classification yard on the Hackensack Meadows, and the freight tunnel to Manhattan, might be made practicable with certain modifications and additions. The writer begs leave to make a suggestion, only, in this direction.

Let there be a great classification and transfer yard in New Jersey. It should be equipped with ample platforms, electric cranes, overhead conveyors, and every known appliance for the economical handling of freight. Connected with it, there should be a series of warehouses—separate warehouses for each class of commodity, such as dry goods, hides and leather, hardware, machinery, furniture, wool, sugar, groceries, wines and liquors, and wheeled vehicles, and refrigerator plants for meat, poultry and dairy products,—all fitted with show-rooms having light, heat, electric transmission, etc.

Break bulk in this yard from the long-distance, heavy cars. Such freight as is not required to enter Manhattan, or can be sold by sample, as is done in Europe, may be stored in a space which will cost enough less than that of the same space on the Island to make a fairly good profit when the saved carting charges are considered.

Such freight as must be carried to Manhattan can be classified, not by 50-ton lots, but by cases, as mail is classified, and loaded on flats similar to those used abroad, with freight for each different section, or each separate concern, by itself, on these light cars.

Run through the tunnel, or tunnels, which need not cost more than one-half as much as tunnels for standard rolling stock, and, in Man-

Mr. Harrison.

hattan, build cross-lines connecting with two or more north and south lines of freight subways, with, say, two running tracks, and continuous sidings on each side, connected to running tracks by switches at short intervals.

In the sidings, at points desired by shippers, place turn-tables, so that a flat can be turned and pushed at right angles into the cellar of a building, and there unloaded and reloaded at the pleasure of the shipper. At convenient points let there be established general stations below the street level. The space above can be used for buildings. Build ramps from the streets for cartage to and from these stations. The stations could be of any size, occupying several blocks. Markets, similar to Smithfield in London, could be established in the same way at points of most usefulness. The gauge of these light lines need not, necessarily, be as wide as that of standard steam roads.

As for the transfer of freight to and from vessels on the Manhattan front, there is no better plan than that now in use, namely, by lighters in the slips outside of the vessels, and handling the freight to and from the hold by the ship's own tackle; no quicker, safer, or more economical plan can be devised.

It might be arranged that much of the coal delivered into large office buildings could be handled in this way, without ferriage and cartage through the streets.

In conclusion, let the City of New York rise to the appreciation of its metropolitan character, as the gateway of a continent, and include, and exploit in its interest, the whole area within a radius of 25 miles from Madison Square.

Let the banking, the buying and selling, the palaces of art and amusement, the town houses of the millionaires, the hotels, be on the Island. Every acre of its surface will be needed for these purposes in time.

Do your manufacturing, handling, and storage of heavy and bulky goods, and house your working people in the outer zones, where you can have a railroad back of the factory, and navigable water in front, and where moderate-priced, civilized dwellings, with light and air, can be built for the people. That way lies the path of modern thought and progress in city building.

State lines, except for taxation, are nothing. Manhattan as the center of trade, capital, amusement, and art, of a community such as this, would not miss the taxes from sweat shops and crowded tenements, and, in economy of doing business and handling commodities, could command the trade of the world.

Mr. Higgins.

Charles H. Higgins, M. Am. Soc. C. E.—This able paper deals with a matter of great local and even national importance. The problem of port facilities is fundamental. New York owes its very being to the wonderful natural advantages of its port. No petty

Mr. Higgins. rivalries or jealousies, as between individuals, companies, or even States, should be allowed to hamper the development of the resources of this port, if it is to be of the greatest service and maintain its present relative position.

A large part of the Port of New York is within the boundaries of the State of New Jersey. There, all but one of the great railways connecting with the productive West, have their terminals. There, freight intended for transhipment could be received directly on the pier without expense for lighterage. It is true that this latter transportation is said to be free within "lighterage limits," but all know that the expense must be borne, whether it is specified as a lighterage charge or is said to be "absorbed" in the freight rate.

As to New York City's daily supplies, they must cross the Hudson, and the solution offered by Mr. Cresson is attractive; however, a great tonnage of goods intended for transhipment reaches the Jersey shore by rail, and must continue to do so. If these goods could be transhipped directly from the piers, an immense saving would be made. No imaginary line, such as a boundary between States, should be allowed to interfere with this natural flow of goods, if the most is to be made of the natural resources of the port.

The North River has two shores. Manhattan Island, on the east, is a long ridge of Archæan rock, covered more or less completely with glacial drift. On the west side there is a similar ridge of like rock lying nearer the surface than that of lower Manhattan and not as deeply covered with glacial drift. This western ridge, extending from Castle Point, Hoboken—where ocean piers already exist—through lower Jersey City, once called Paulus Hook, in turn appears at Ellis Island, Liberty Island, Robins Reef, and near Constable Hook. It is true that this ridge, in the intervals between the prominent points mentioned, is or was covered by a few feet of water, but the old maps show the extreme low-water line extending along almost the entire distance; and, as said before, the rock, which in lower Manhattan is from 60 to 100 ft. below the surface, is found along this ridge at less than half that depth. It is this ancient metamorphic rock which, in this locality, must be reached in order to support heavy structures. This ridge, with deep water or soft, easily dredged mud along its easterly side, is a great natural resource which should not be neglected because of a State boundary line, or because it is covered with 10 ft. of water instead of 10 ft. of sand, and it is for engineers to point this out. To laymen, perhaps anything above water is solid land and anything else is a natural waterway. The method of showing the land and water masses on maps tends to deepen this impression. Engineers, however, who design and build foundations, know that this is not the economic division. The depth, to a stratum capable of carrying great loads safely, is a controlling element in the cost of development.

Mr. Higgins.

This is not a discovery. For years, it has been known to engineers, and some others, and the thought can be traced back through scientific papers and reports for many years; but its significance has not yet been brought home to the public. This is due, largely perhaps, to the State boundary line, coupled with the fact that, until recently, Manhattan Island has offered the necessary facilities.

To bring about this development, the Federal Government must take a hand. Harbor lines must be changed, and new policies originated. New York's true advantage lies in this direction, for only by seizing all the natural advantages of the harbor, can it maintain its true position as a port.

Mr. Lewinson.

M. Lewinson, M. Am. Soc. C. E. (by letter).—Mr. Cresson's meritorious proposition to relieve the congestion of traffic at the New York piers and provide accommodations for the new transatlantic steamers, which require longer piers, has only this disadvantage, that New York City, being already overburdened with taxes, may not, for some time to come, be able to raise the money for such an undertaking. The delay in building the subways, so greatly needed, is caused by the same lack of funds. Furthermore, the condemnation proceedings, necessary in Mr. Cresson's plan, are tedious, very long, costly, and out of proportion to the real value of the property taken over. Nevertheless, New York City must have more piers and longer ones, in order to hold its commercial supremacy, and it seems to the writer that the project advanced by T. Kennard Thomson, M. Am. Soc. C. E., to extend Manhattan Island, should be given more serious consideration than it has received heretofore.

As the writer understands it, Mr. Thomson proposes to give to the city, free of cost, all the water-front of the newly made land, meeting the expenses of such an undertaking by the sale of lots along the new streets and avenues. That would add about 9 miles of water-front, which would be available for the building of new piers, and an immense revenue would be derived from them as well as from the taxes on the new land and buildings.

As this plan does not narrow the main channel, which is kept of the same width as The Narrows, and as the difference in the tide would be imperceptible, the writer is of the opinion that the War Department would not object to it, and the acquisition of Governors Island would not be difficult, as Congress is considering the sale of this island to the city.

By building the elevated railway along the westerly side of Manhattan Island, as proposed by Mr. Cresson, connecting it with the railroad subway proposed by Mr. Thomson, and further connecting this subway by a tunnel to Staten Island and thence by rail and tunnel to New Jersey, most of the advantages proposed by Mr. Cresson would be achieved.

Mr. Thomson.

T. KENNARD THOMSON, M. AM. SOC. C. E.—Mr. Cresson and Commissioner Tomkins deserve great credit for the way in which they have presented this subject, and for the great amount of honest hard work that they have done toward solving the very serious, in fact, the vital, problem of handling freight in New York City; for, it must be patent to everybody that something must be done at once if New York is to maintain the position it should. If it is impossible to supply the demand for docks now, the condition must become worse every day, even before the opening of the Panama Canal, the Barge Canal, and the Intercoastal Canal, for any of which New York is utterly unprepared. Europe, South America, Boston, Philadelphia, are awake and spending millions, while New York sleeps.

For the past 15 years the speaker has never lost an opportunity to point out that Manhattan is no place for a terminal of any kind—either freight or passenger—and on February 6th, 1898, he published* a plan to solve the Brooklyn Bridge terminal problem by simply abolishing the Terminal and continuing the line to West Street, thence northward, turning to cross the Williamsburgh Bridge, making a wide circuit in Brooklyn and then connecting the Brooklyn end of the old bridge. (Fig. 7.)

If this had been done, with practically an endless chain of cars running in both directions, connection could have been made at once with all north and south lines in Manhattan, and passengers could have been transferred readily from the Bridge loop to the line desired without being forced to get off the bridge cars at points inconvenient for nearly all. The subsequent Bridge loop, which has been built but not yet put in operation, will not accomplish the desired result, as most of the passengers will still have to leave the train before reaching their destination and then take a long walk. The speaker's object in citing this is simply to give an example, known to all New York, of a very bad situation in the passenger terminal line, and if it is so hard to handle passengers in a terminal, will it not be much harder to handle freight?

Mr. Cresson has shown the congestion of freight all along the water-front, trucks waiting their turn to reach the docks. Even allowing for systematic handling, will not the congestion on the streets become worse when, instead of distribution at every dock, the bulk of the freight is bunched into a few terminals, as he suggests?

Another objection to Mr. Cresson's plan is the fact that freight from Philadelphia to Lower Manhattan must go many miles out of the direct route; this is unnecessary, as will be shown later.

Still another objection is that the number of tracks which would be required would make this roundabout route impossible on account of the cost. It would be interesting to have a railroad man work up

* In *The Brooklyn Eagle*.

Mr. Thomson.

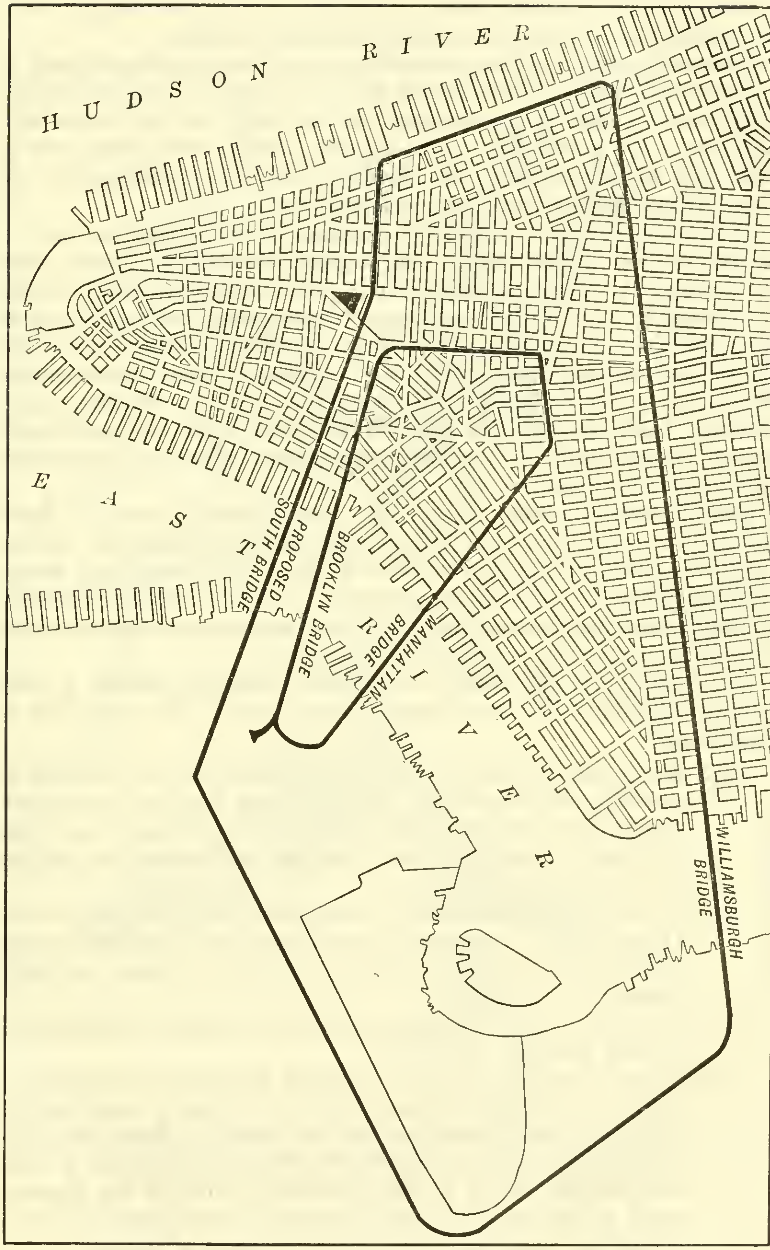


Fig. 7.—The Inner Circuit Shows an Inadequate Belt Line; the Outer One an Adequate Belt Line.

Mr. Thomson. a train schedule showing how the requisite quantity of freight could be handled, and stating the number of tracks required.

A vice-president of one of the big trunk lines has stated that, if the speaker's plan of freight belt lines of tracks around Manhattan and Staten Island is carried out, at least eight or ten tracks will be kept busy. A belt line could certainly handle much more than a dead end with terminals. Mr. Cresson's plan does nothing for the East Side, unless by trucking across the city.

The speaker would respectfully suggest to Mr. Tomkins that, to make the Dock Department's plan feasible, he put his tunnel near the Battery and run an 8-track freight road around the entire Island of Manhattan, having the tracks on the street level inside of the dock line (off the street), if possible, with an elevated street above the tracks (as has been suggested by Mr. McBean), and tunnels under the tracks at each ferry for automobiles and trucks, the passengers crossing overhead. By this plan, freight cars could be readily unloaded at any point of the water-front, thus affording distribution instead of concentration.

Mr. Cresson has cited the Pennsylvania Railroad Tunnel to show that his scheme is feasible, but nobody denies that tunnels can be built under the North River; however, from all reports, the Pennsylvania Tunnel is not a financial success, and, if used as a criterion for building a freight terminal, the shippers and other victims would probably object to the great burden of expense.

In April, 1911, the speaker submitted to Mayor Gaynor a plan which would afford immediate relief in many ways. This plan, Fig. 8, consisted of:

1st. Extending the Battery 4 miles into the Bay by building a new city, 10 blocks wide and 80 blocks long, or 800 square blocks; thus adding to Manhattan 1 400 acres and more than 8 miles of new docks, including dry docks, sites for public buildings, etc.

2d. Connecting Staten Island to Manhattan by an 8-track tunnel, $1\frac{1}{4}$ miles long, thus giving the Pennsylvania, Baltimore and Ohio, and other railroads the most direct entrance to Manhattan.

3d. Connecting South Brooklyn with the Manhattan extension by another tunnel.

4th. Constructing freight tracks around Manhattan (6 tracks at least) and around Staten Island (4 tracks to start with), and also freight tracks around the Brooklyn shore, with the great benefit of an arrangement which would permit a full-sized freight car to be taken promptly, and by the shortest route, to any dock in New York and at once unloaded without having made any unnecessary 20 miles of detour.

Mr. Thomson.

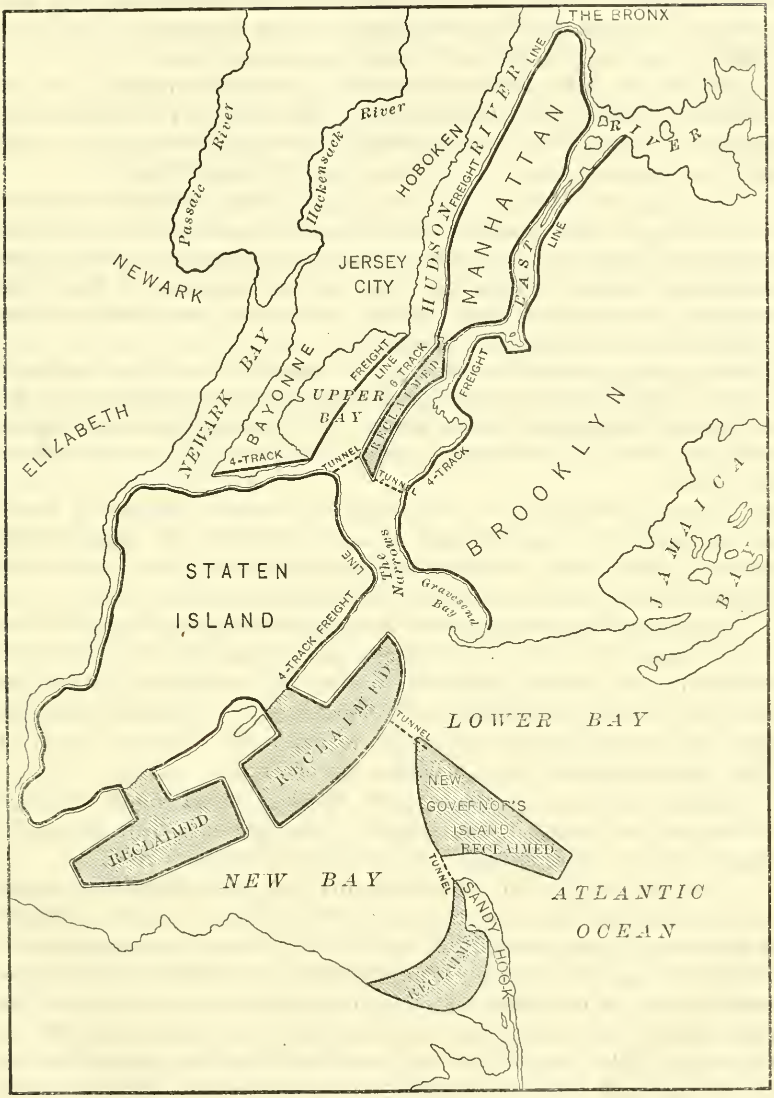


Fig. 8.—A Solution of the New York Harbor Problem.

Mr. Thomson.

Thus, without being asked for a cent of money, the City could obtain 8 miles of new sea walls for docks, and increase the taxable value of Staten Island from \$50 000 000 to \$500 000 000 and also levy taxes on at least \$1 500 000 000 worth of new property; or, in short, collect more than \$50 000 000 of new taxes every year.

It has been claimed that these great improvements might depreciate the values of land on Manhattan; but how could any plan which involved the expenditure of at least \$50 000 000 every year in this locality help but improve every foot of New York City and State?

It would take a Jules Verne to imagine all the benefits that would accrue, among the lesser of which would be abolishing car-floats, preventing interfering currents at the Battery, saving 2 hours' time for ocean steamers, putting the City Hall in the center of New York instead of at the end; and, in fact, everything connected with the building of a model "City Beautiful."

It might be stated here that, after sending this plan to Mayor Gaynor, Mr. Cresson showed the speaker the plan, published by Mr. H. Arnold Ruge some years before, for joining Governor's Island with the Battery of which the speaker had not heard prior to writing the Mayor.

The speaker's first idea was simply to connect Governor's Island to the Battery, but he discarded it as useless when the idea occurred to him, weeks later, of continuing the extension for 4 miles until Staten Island could be reached by a reasonable tunnel.

Nobody wants to build or ride in a 5-mile tunnel from Staten Island, but a tunnel $1\frac{1}{4}$ or $1\frac{1}{2}$ miles long would be satisfactory. In short, permission to extend Manhattan could not be obtained unless the resulting benefits to the whole city were great. The speaker believes that, when these benefits are fully realized, the City and State of New York will never rest until they are obtained.

Where rock can be reached, the speaker would simply build a coffer-dam and pump it out, instead of back-filling; then the building of subways, sewers, pipes, etc., would be simplicity itself.

After the speaker had formulated this plan, he realized that within 5 years after its completion this Manhattan extension would be entirely inadequate to take care of the growth of the city, the population of which will probably double (from 6 000 000 to 12 000 000) during the next 30 years, if not restricted; so he developed the second part of this plan, though he did not deem it advisable to do more than refer to the latter.* This second part has never been shown to any one outside of the speaker's office (except two men), and is now brought before this Society. It is shown on Fig. 8, and the principal features are:

* *Engineering News*, May 11th, 1911.

1st. Build an island (New Governor's Island) near Sandy Hook, to contain about 8 sq. miles, in such a way as to form a new bay, much larger than New York Bay. Mr. Thomson.

2d. Reclaim many acres on the inside of Sandy Hook.

3d. Reclaim two strips of land and connect them with Staten Island in such a way as to form the maximum amount of fine harbors for docks, etc., making in all about 30 sq. miles of valuable property for shipping, manufacturing, and other purposes; and

4th. Connect all these lands with tunnels.

The Government could then give up the Brooklyn Navy Yard for a much better one; give up Governor's Island for a much finer site; and the City would be able to take care of the traffic of the Barge Canal, the Panama Canal, the proposed Intercoastal Canal, and any other sudden increase in shipping, instead of being continually obliged to refuse requests for docks and thus force the shipping to anchor in the bay for days at a time, which, of course, is very expensive.

The Lower Bay proposition should not be carried out before the Upper Bay portion, but work could be started on it as soon as the latter is completed.

Of course, the scheme for deepening the East River, proposed by W. M. Black, M. Am. Soc. C. E., should be carried out, but the City should not waste money which it cannot afford on the many other schemes submitted, involving many millions of public money, as their usefulness is very debatable when a project like this can be handled by private enterprise. This plan would afford quick relief and yet be capable of further development for 50 years to come, 100 years being too far ahead for the speaker's vision.

Mr. Tomkins.

CALVIN TOMKINS, ASSOC. AM. SOC. C. E.—The speaker will consider this subject from its larger viewpoint, and not from the technical side. Under the direction of Mr. Cresson, and with the assistance of Messrs. Staniford, Hoag, Betts, and other engineers of the Dock Department, to whom great credit is due, the details of this plan have been carefully worked out.

The port problem of New York is not local. The Dock Department is regarded by the people of the City, and largely by the officials, as a kind of fifth wheel in the Municipal Administration; it is considered as important, but about it they know very little. It is removed from the immediate touch of their daily existence, and they do not appreciate the magnitude or difficulty of the problem. They do not realize that the cheap handling of commodities in and out of the Port, on both sides of the Hudson River, is the factor on which rests the continuous, orderly, progressive development of the City. It is a national problem. It is in effect an international problem; but it is

Mr. Tomkins. so mixed up with local affairs which, in the eye of the citizens, are of more immedate importance, that it does not receive the attention it should.

On the completion of the Panama and Erie Canals—and the possible termination of the freight discrimination against the City of New York—the Port's new responsibilities cannot be measured by those of the past. Mr. Hoag has said that nobody, even a few years ago, could foresee the size of modern ships; but, after all, it is the growth of the City which is most responsible for the pressure on it; and we are confronted with the problem of managing the commerce of the greatest port in the world.

New York, at present, is second in size only to London, and is first in importance as a seaport. In former times, Venice, Amsterdam, and London, each in turn, became the world's great ports for international exchange. Commerce was diffused between English and North Sea ports during the last century, and there was no one great world's seaport such as New York will be on the completion of the Panama Canal. This is the responsibility which confronts the City, and for which, during the last three years, it has done little to prepare itself. It has maintained existing docks in a good state of repair, but has done almost nothing in the way of new construction.

In former administrations, the responsibility was recognized to some extent, and it is not fair to state that the City has not had a very comprehensive port policy, as far as it could see ahead. In 1871, New York, in advance of all American seaports, began to municipalize its water-front. It has made the most of its water highway along the East and North Rivers, and, until Messrs. Jacobs, Davies, Forgie, and their associates demonstrated the practicability of passenger tunnels under the Hudson River, there was nothing to depend on but the water highway and transmission by car-floats over that highway. With the advent of tunnels, however, this water-highway idea may be considered somewhat in the nature of a traffic tradition, as it is not the most economical method of transporting freight around the harbor, though undoubtedly, the car-float will be used for outlying Boroughs and minor terminals after tunnels have been built. Before the advent of freight tunnels, dependence was placed on car-floats altogether. That direct, continuous, all-rail connections are the cheapest, is shown: by the efforts of the New York Central Railroad to come down from Spuyten Duyvil to 60th Street and thence to Lower Manhattan; by the construction of the Pennsylvania passenger tunnel from New Jersey across Manhattan to Long Island; by the construction of the Pennsylvania Bridge across Long Island Sound; and by the prospect of a tunnel to complete the route under or near the Narrows. The tendency is away from intermittent car-float transmission and toward continuous tunnel or bridge transmission.

Mr. Tomkins.

The port problem, stated very simply, is the proper co-ordination of railway terminals, and is not at all novel. Briefly stated, it is to put each part of the harbor to its best natural uses, and plan with the expectation of subsequently connecting all the parts through the instrumentality of tunnels and floats; that is all there is to it. Fundamentally, it is a question of co-ordinating and bringing together the various railroads about the port.

In Bremen, Antwerp, and Manchester, there are systems of loop lines of railroad connections, making every part of the port available to every other part, so that the cars from any railroad have access to all parts, with no expensive transfer charges to factories or warehouses.

It must be remembered that New York, in addition to being the greatest commercial city in the world, is also the greatest manufacturing city in North America, and its prosperity depends on its industries more than on its commerce. Commodities passing through the city, or going into warehouse or terminal and being brought out again, are of little local importance in comparison with the raw materials which are brought here to be wrought into finished products and sent out to the markets of the world, thus increasing the revenues of the City. It is of more importance that every factory in the City should have a railroad siding connecting it with the ocean terminals of the Port and with every railroad, so that it can find ingress and egress for its raw materials and products.

The problem is not so much a steamship as a railroad problem. It is not novel, as has just been stated. The City is not making any radical departure, but is simply following the well-trodden path of experience along which the great ports of the North Sea have advanced, except that local conditions are being adjusted under the guidance of that experience, and such changes are being made in local plans as local conditions make necessary. The ultimate idea is that of joining all parts of the Port, including the elevated railroad on West Street, with tunnels to New Jersey and Long Island, and utilizing the New Jersey meadows for distribution and classification yards.

One of the difficulties in the organization has been the magnitude of the Port itself; such an embarrassment of opportunities does not exist anywhere else in the world. The difficulties of the present time will be most beneficent influences in the future, when they have been conquered and harnessed to our needs.

The Port is divided into four grand sections by the harbor waters: New Jersey on the west, Long Island on the east, Manhattan and the Bronx at the center, and Staten Island on the south. It is difficult to connect these parts by tunnels. It is impossible to do so at once; it must be done gradually, keeping a policy in mind and following it up.

Mr. Tomkins. The jurisdiction of the Port lies in two States, which, at the present time, is a difficulty; but the appointment by the Governors of these States of two Commissions, which are associated, is a long step toward solving these interstate complications. In the long run, the speaker is convinced that the interstate character of the Port will be an advantage and not a disadvantage, because there must be interstate regulation by the Federal Government over the terminals at the Port, before there can be unity of administration and unity of control over rates and terminal charges, to make the Port most effective.

The greatest difficulty is the tradition among the railroads of utilizing terminals for competitive purposes. Competition among the railroads, since the advent of the Interstate Commerce Commission, has been confined to that at terminals. This is all that is left of the drastic competition of former days. The railroads have established extensive terminals to attract freight to their lines. In competition with each other, they have been obliged to do this. The richer and more enterprising lines have secured the best terminals, and they are naturally loath to give up their advantages to other railroads.

The organization of this Port, like those of Antwerp, Hamburg, or Manchester, involves the abandonment of this individualistic advantage, and means that freight collected at any part of the Port, or sent over any railroad to the Port, should enjoy the use of all the terminals. The poorest and the least enterprising railroads would thus have about the same advantages as the richer and more enterprising ones, and that, naturally, is not acceptable to the latter. Not only at New York, but at Baltimore, Philadelphia, and Boston, the effort is being made to co-ordinate the railroad systems and substitute for expensive transfer charges a cheap switching charge. At present almost all the endeavors of Baltimore and Boston are being directed toward this end, rather than toward the creation of a great physical system of docks, which is not needed. It is peculiarily difficult to do this in New York, because there are so many railroads, with such intense rivalry between them; but, under the force of public opinion and business necessities, this is coming.

The City was given power by the last Legislature to administer the Port and create terminals in a most comprehensive manner. The law has clothed the Dock Department, under the Board of Estimate and Apportionment, with perhaps dangerously large powers in this respect—not only to take the water-front and make terminal improvements on it, but to take the lands back of it and organize them; to build railroads and warehouses, and, if necessary, to operate them; and even to take lands, impose a plan upon them, and sell them or lease them after the plan shall have been imposed. That bill gives the City the power of excess condemnation in the Dock Department, which no other city enjoys. The speaker believes that, if it can be shown that the plan is a public necessity, the Courts will sustain the law. Mr. Tomkins.

There is also power under the law to create or permit the creation of private terminals, as distinguished from railroad or manufacturing corporations—a new kind of corporation having certain limited powers—so that advantage can be taken of private capital and private experience as well as those of the municipality.

The speaker's idea of an improvement policy is that the City should plan the physical construction, make the land acquisitions, and then create terminal corporations and turn over the administration to them—either to railroad or terminal corporations—taking advantage of private capital, ability, and experience, rather than saddle the City with all these responsibilities at this time. The City should now adopt a large and comprehensive plan for future development.

Mr. Cresson has called attention to the principal factor in that plan, the West Side terminal extension; but the Dock Department has plans for South Brooklyn, which involve a larger extension and are bound to come soon; plans for Staten Island, the Bronx, and the East River; for Newtown Creek, which Mr. Forgie mentions but does not enlarge on; and for all the various parts of the Harbor.

Next to the Panama Canal, the organization of the Port of New York is probably the greatest physical undertaking that the world has before it at the present time. This is not an exaggerated statement, and the speaker does not believe that the Dock Department has solved the problem finally; but it has made an attempt to do so; it has, at least, taken the responsibility for the attempt. The plan is there, for what it is worth, subject to such criticism and modification as may be found necessary. The speaker wants the best plan that the City can get; and, in relation to the West Street plan, hopes it will be criticized. Certainly, there are none more competent to do that than the members of this Society.

The West Street plan is the key to the whole situation. Until that is settled, the Dock Department cannot tell what to do with the big ships; whether they are to come to Manhattan (where they want to come with their passengers and freight, for the same reasons that the railroads want to come), or whether it will be necessary to send them to South Brooklyn, if they must be excluded from Manhattan in order to provide for the railroads. The City's policy should be made known quickly.

The railroad traffic on the West Side of Manhattan must be taken care of first. That is a fact which has come home to the speaker with greater force since he has been Dock Commissioner, than it did before. The impression has grown that the railroads can be removed or restricted in their operations at the water-side and steamships put in their

Mr. Tomkins. place. This cannot be done until equally good or better facilities are provided for them on the land side of the street.

The railroads bring the food supplies to Manhattan, and take away the package freight, and they must be permitted to remain at the dock front until better provision is made. This is the way to solve the problem—the only way, the speaker believes—and if this plan is not adopted at once, it will be later, because there is no other way.

The reason is that it is impossible to cut the tracks of the New York Central Railroad. It would be like cutting the throat of the City of New York, commercially speaking, for this railroad has the only direct, continuous, all-rail line into the city. There are many lines coming from the West to New Jersey, but there is only one line coming to New York City. The City cannot cut the tracks, in conformity with the suggestion made by a majority committee report of the Board of Estimate and Apportionment, and close the traffic. The Dock Department wants to discontinue floating from 60th Street that large portion of the products which comes over the Central lines now. It wants an up-to-date all-rail route into Lower Manhattan, instead of the slow and expensive transit over surface tracks. A large part of the New York Central's freight comes by car-floats from the 60th Street yard; and if the road is cut at 30th Street or 60th Street, there will ensue a still greater movement of car-floats and a still greater congestion at the dock front of Lower Manhattan. Therefore, it is a question of getting the tracks off the surface of the streets, and more conveniently located. Provision must be made for a more continuous movement. The Courts have recognized the franchise rights of the New York Central Railroad; and they say it cannot be kept out of Manhattan; but the method which that road shall use to bring its freight down can be imposed by the City, if its conditions are reasonable. That is what the Dock Department is endeavoring to provide—reasonable conditions for taking care of that traffic—thus relieving the congestion on the water-front and on West Street. As Mr. Forgie has stated, the present method is not civilized; we are past that stage.

It can be done in either of two ways: through the instrumentality of an elevated railroad, or by a subway, and the speaker has no predilection for either. The engineers of the Dock Department have examined into the matter very carefully, and have come to the same conclusion as Mr. Forgie—that a subway is impracticable on account of its cost of construction and operation, and its danger. It cannot be brought below 30th Street. Its construction would create a great disorganization of traffic; it would be dangerous to operate; and the set-offs which are so easily obtained by the elevated railroad could not be obtained by the subway without great expense, risk, and danger.

The last choice is a marginal elevated freight way. Shall it be a New York Central enterprise, or shall it be available for the other roads as well? The very best and cheapest facilities must be provided for this road; interference with its traffic must not be permitted, because it is expected that it will act as the pace-maker and the rate-maker for the other railroads. By creating the very best conditions under which it may operate, there will be created the conditions under which the other railroads must also operate. If this road comes down over this elevated railroad, and the others are prevented from using it, that road will dominate the freight situation of the West Side of Manhattan absolutely. The other roads would not be willing to submit to this. Already their unfriendly attitude has changed, since the Department has shown a way of enabling them to use it by tunnels, under as favorable conditions as the New York Central will enjoy. Mr. Tomkins.

When the railroad is built and the Central uses it, the other roads must use it. It is not intended to compel them to use it. As Mr. Harding has said, when the railroad managers find a cheaper and better way of doing their business they will do it that way; and if the opportunity is held open to them to use the elevated freight facilities they will take advantage of them. The erection of this elevated railroad, and holding the opportunity open to the other railroads, means the eventual abandonment of the dock front by the railroads, and the opening of the docks to the uses of steamships, for the railroads will have indefinite room for expansion on the land side of West Street. The whole situation will thus clear up in Manhattan. This cannot be done at a blow, but must be a gradual, orderly procedure.

The one criticism of the City Government, from the Dock Department standpoint, if there is any criticism to be made, is that it has not acted promptly in this matter. Public interest has been so absorbed in subway and police matters, and in sundry measures of very great local importance (though the speaker does not wish to minimize their importance in any sense whatever), that this matter has not been given sufficient attention, and the consequence is that action has been delayed. The New York Central, under the latest legislation, was obliged to submit its plans on October 1st, 1911. It complied with the law. At the speaker's request, its plans were then referred to the Dock Department, and he took them up with the engineers, and one month thereafter the Department submitted plans. Physically, these are very much like those of the New York Central, but differ from them in providing that the elevated road shall be a public highway below 60th Street, giving the Central such rights as it requires, and reserving running rights for the other railroads.

As the Central officials will undoubtedly stand out for the best terms they can get from the City, and the City will try to get the best bargain it can, an immediate conclusion cannot be expected. The speaker does not think that any legislation will be necessary to compel the Central to accept a reasonable plan. He believes this plan to be a

Mr. Tomkins. reasonable one, and that in all good faith the differences that may be found between the City and the Central can be adjusted through negotiation rather than through legislation; but the City has lost much time since November 2d, 1911, in refraining from negotiation.

As Mr. Cresson has said, the Dock Department is not seeking to bring business to Manhattan; in fact, the whole purpose of its plans is to create terminals sufficiently attractive in outlying boroughs, so that manufacturers will go away from Manhattan. We are seeking to modernize the New York Central terminals in Manhattan, and make more room for transatlantic and coastwise steamships and their passenger business.

The speaker would like to reiterate: If there are any criticisms of these plans, to be made by any members of this Society, he would like to have them as soon as possible.

Mr. Moore. ERNEST C. MOORE, ESQ.—The freight-handling problem of the West Side of New York City has received constant attention for the past twenty years. Many of the leading engineers of the United States have been called in consultation, and many committees from the leading business organizations of the City have been appointed at various times for its consideration, and though many reports and recommendations have been made, nothing has yet been accomplished.

The urgency for the settlement of the question has increased as time has passed, until now it has become absolutely necessary that an early conclusion be reached. Some of the most urgent matters are:

First.—The necessity for the removal of the freight tracks of the New York Central Railroad from the surface of Eleventh Avenue.

Second.—The necessity of providing terminals for the Barge Canal along the North River.

Third.—The organization of the freight-handling business of the railroads on the North River. At present freight is handled from car-floats over the piers and by trucks through the streets. The railroad facilities for doing this work are now taxed to the utmost, and business is still increasing.

Fourth.—The necessity of providing additional facilities for the steamship business. Many applications are on file in the Dock Commissioner's Office for increased pier space for steamers; and this business must either go to some other place or have more facilities provided here.

With these facts in view, the Board of Estimate, in July, 1910, authorized the appointment of a committee of engineers to pass on the engineering features of the situation. The speaker had the honor of being one of the members of that Committee.

Mr. Moore.

In considering this problem, the Committee had before it the various plans and recommendations which had previously been made by other committees and engineers who had studied the situation. In order to get all the facts with regard to present conditions, the Committee first called a meeting at which the attendance was requested of an officer of each of the various railroads serving the city. At this meeting one vice-president of each railroad was present; a great deal of confidential information was asked and obtained as to the quantity of freight handled, the cost of handling, the amount of increase in the past ten years, etc.

It was the desire of the speaker and the other members of the Committee that a unanimous report be rendered and that the Committee be able to recommend a plan which would be acceptable to all parties and be adopted.

It was finally concluded that one of three plans must be adopted: A subway; an elevated railroad along West Street; or the construction of separate terminal stations on the east side of West Street to be connected with the piers by a bridge over the street and a ramp down to the pier level.

The Committee agreed that the problem involved was that of getting the railroad freight business into terminal stations on the east side of West Street and relieving the water-front as much as possible of railroad occupancy.

It was further agreed that joint terminal stations were desirable from the standpoint of the City's interest, and it was also thought that this plan would be to the interest of the railroads. With this point, however, the railroad officials disagreed, because, owing to the workings of the Interstate Commerce Commission, competition between railroads in handling freight has been largely reduced to promptness of delivery and competition in the terminals. This competition might disappear in a joint terminal, or favoritism might be shown. There are a number of joint terminals in successful and satisfactory operation, both to the shippers and the railroads, however, and it seemed to the Committee to be only a question of management.

As to the best method of bringing about these results, the Committee unfortunately could not agree. With regard to the subway it was agreed that it would be impracticable below 30th Street, but two members of the Committee recommended the construction of a subway down to 30th Street with the so-called unit terminals below that point. The speaker could not join in the recommendation as to the subway, because it appeared to him that it would cost at least three times as much as an elevated railroad; that it would not serve the shippers so well; that it would be difficult of access; that it could not be connected with the piers; that as it would be very expensive to connect with buildings, its utility would be much less; and that it would give

Mr. Moore. permanent possession of Eleventh Avenue to the New York Central Railroad and forever block the reorganization of the freight business above 30th Street.

With regard to the unit terminal stations, their adoption did not seem possible. A large number of the railroads terminating in New Jersey are seriously in need of additional facilities on the east side of the river, and the privilege of building these terminal stations was offered to each railroad with the assurance that, if acceptable, the Committee would unanimously recommend their construction to the Board. Even under these conditions, and after a very considerable time had elapsed, neither the New Jersey railroads nor the New York Central Railroad would agree to undertake any such construction, even if they had the privilege. The reason is not hard to understand.

The construction of one of these unit-terminal stations involves an investment of about $10 000 000. It would require almost as much space as two piers, and, with the possible exception of one or two places on the North River, could not be constructed without destroying at least two piers, so that a railroad company, in undertaking this construction, must lose the use of two of its piers for at least one year and perhaps two, and it must make an investment of about $10 000 000 without the least assurance that the new equipment would increase, even to the very slightest extent, its handling capacity over that enjoyed before the reconstruction of its piers. This is hardly a proposition which any railroad could be expected to accept. Furthermore, if such an installation was built, it must necessarily be for the use of a single railroad and would not be a joint terminal, as the piers at which these constructions would have to be made are now controlled by the railroads, and no railroad would accept joint occupancy with another railroad on its own property. The limited space between the station and the pier is an insurmountable obstacle to the successful operation of this plan, and in view of the fact that, in the available space, a railroad operation over the numerous sharp angle switches and steep grades involved in the plan proposed cannot be carried on without great difficulty and expense, the capacity of such a station must be very much restricted.

Looking at the matter from another viewpoint, and assuming that the switching layout will supply the station to its full capacity, it seems fair to compare it with that of other stations in actual operation. The capacity of St. John's Park Station, of the New York Central Railroad, is 350 000 tons of freight per annum, and this station is worked 24 hours per day. It is true that its capacity might be considerably increased by mechanical devices, but, after all, the capacity of a station is determined by the area of its loading platforms and the length available for teams.

A comparison of those items is as follows: The area of the team platforms in St. John's Park Terminal is more than double those of the Unit Float Terminals. In the former the team frontage is somewhat greater than that in the latter. St. John's Park has about 50% more team tracks than the proposed Unit Terminals, and the area of one floor alone is nearly 50% greater than the two floors in the proposed Unit Terminal combined. Mr. Moore.

If, therefore, there is assumed for the Unit Float Terminal a capacity of 350 000 tons per annum, it should seemingly be a liberal allowance. The capacity estimated by the Committee for such Unit Float Terminal is 1 500 000 tons, or from four to five times greater than what would seem probable. This estimate makes no provision for future expansions, being for the full capacity of the stations proper and the actual present tonnage.

The handling of the present traffic of 5 000 000 tons by the various railroads in Lower Manhattan, as stated by the Committee, therefore, would require fifteen such terminals; or, as the unit terminals occupy more frontage than the average city pier, a space equivalent to approximately 5 775 lin. ft. of water-front, or the equivalent of approximately twenty-five city piers. The number of terminals required for the district below 30th Street, as estimated by the Committee, is nine, occupying one pier for each terminal, or a total of nine piers such as are now used by the railroads. At the Barclay Street Station, the New York Central has two piers, and the amount of freight handled over these piers is almost exactly the same as that handled at St. John's Park.

Considered from the standpoint of the City, the proposed plan meant a permanent easement to the Railroad Company, forever blocking any harbor improvement or modified use of the docks which future conditions might require. It would require the construction of a number of transverse elevated structures across West Street, thereby cutting off permanently its use for the construction of a longitudinal elevated railroad, either for passengers or freight; and as there could be no assurance of any increase in the capacity of these structures, the City would have no assurance of any relief whatever from pier congestion on the North River, which is one of the principal objects of a re-organization of the freight-handling situation.

Another matter of vital importance adverse to the unit terminals is the fact that the river bottom between Christopher and 46th Streets is of such a nature that piers constructed in it are continually settling. This would be fatal to the operation of cars and electric locomotives over the intricate system of switches which would be necessary on the piers.

On the other hand, it seems that the elevated railroad, which is the plan proposed by Commissioner Tomkins, would meet every one of these requirements. The Jersey railroads could be connected

Mr. Moore. directly with the elevated railroad by tunnels under the North River, and their pier occupancy could be very largely, if not entirely, done away with. Switches from the elevated railroad could reach the piers on one side of the street and the warehouses on the other, and if this plan was recommended to the Board and adopted by it, the New York Central Railroad stood ready to build the structure at once, either for its own use or, through agreement with the City, for the use of all roads; and this it seems must be the construction which will be carried out eventually in the solution of this question.

Mr. Forgie seems to think that New York is not much of a manufacturing city, but is more of a financial city. The following figures from the Census Report of 1905 will tend to disprove this:

"New York is not only the greatest commercial city of the country, but it is also the greatest industrial city. According to the census of 1905, there were in this City 20 839 manufacturing establishments, nearly one-tenth of the entire number in the United States; these had a capital of $1 042 946 487, constituting over 8 per cent. of the total industrial capital of the United States; they employed 464 716 wage-earners, who, with their families, constituted over 50 per cent. of the City's population; there is paid in wages $248 128 259 a year to these workmen, a sum equal to the entire internal revenue receipts of the United States. The total value of manufacturing products in New York City in 1905 was $1 526 523 006, a sum almost exactly equal to the total foreign commerce of the port of New York in 1909, and amounting to 10.27 per cent. of the total value of manufactured products in the United States.

"There are more manufacturing establishments, more manufacturing capital, and more value of manufacturing products in New York City than in any State in the Union, except the State of Pennsylvania, and, of course, the State of New York, of which this City is a part."

The records of the Building Department show that permits are issued annually for the construction of 1 000 new factory buildings in the Borough of Manhattan alone.

Mr. Bolton. REGINALD PELHAM BOLTON, M. AM. SOC. C. E. (by letter).—Mr. Cresson's paper is a very clear and interesting re-statement of the arguments which for some time past have been advanced by Calvin Tomkins, Assoc. Am. Soc. C. E., Commissioner of Docks. These arguments, while they very properly and clearly describe the existing deficiencies and inconvenience, do not, in the judgment of those equally entitled to express opinions on the subject, offer solutions which would have the effect of immediate and future relief, without permanent injury to the future interests of New York City.

The proposals made by Mr. Cresson, as well as by the Commissioner of Docks, seem to be based on a limited conception of the possibilities of other means of relief, and mainly on preconceived assumptions that the trade and traffic of the West Side must be handled in a manner similar to that of such institutions as the Bush Terminal, or of other ports where the conditions are totally different from those now obtaining, or in the future to obtain, on the West Side of New York City. Mr. Bolton

The ordinary port terminal, such as in Montreal, New Orleans, etc., is a place of wharfage for vessels to which direct railroad communication is very properly provided, but the dockage of ships along the margin of the Island of Manhattan is not of the same class of wharfage, and the business handled is not of the same character.

It is evident that the West Side of Manhattan is destined to accommodate large passenger steamship traffic, the freight of which is mainly of the package description, and that the relation of railroad-borne freight to this traffic can best be provided by water carriers. The purpose of a connecting freight railroad is not evident from the conditions, either as laid down in this paper, or as disclosed from examination of the actual situation.

It is difficult to find any fundamental value in the proposition of a marginal railway, more especially in the form proposed, of an elevated structure to which cars have to be hauled by switching and grading from car-floats, or raised by excessive grades from long under-river tunnels, as proposed in the Dock Commissioner's scheme.

The confusion likely to arise by cross traffic, from the numerous points of entry of the cars, can readily be conceived, and also the condition to which the West Side would be reduced by the drilling into position of a large number of cars for passage to and from one or other of the assumed terminals. What saving would there be, for instance, in hauling a freight car from the vast yard in Brooklyn, proposed by the Dock Commissioner, down under the East River, up again to Manhattan along the west front, to some point to be transferred into the terminal to which it could have been much more readily brought by a car-float directly from its original position.

The whole scheme of a marginal railroad back of the docks is evidently unsuited to the West Side conditions, where the object of the handling of freight is its transfer to or from the street truck.

A far better scheme is that proposed by W. J. Wilgus, M. Am. Soc. C. E., for a freight tunnel of small section, and of similar character to the Chicago freight tunnels, extending not only along the West Side, where its application and value would be restricted, but into the interior of Manhattan.

The scheme of a marginal elevated railroad is destructive of the hopes and necessities of the public in the matter of passenger transit along the important lines of access which West Street affords, and the interests of the Port will be far better served by a passenger railway, transferring passengers, baggage, and express freight, than by any scheme for handling the cars of railroad companies.

The immediate difficulties with which the Dock Department finds

Mr. Bolton. itself confronted can best be met by a prompt provision of suitable equipment for the more rapid handling of freight in and out of cars on car-floats. The necessary terminal facilities, which Mr. Cresson proposed should be provided on private property on the east side of West Street and the marginal way, can be provided just as well, and to much better purpose, on city property, on the piers themselves, where the present use of this valuable space is of the most limited and restricted character.

In place of using twenty-nine piers along the West Side by railroads, it is evident that proper interchangeable terminal facilities could be provided, as proposed by the Special Committee of the Board of Estimate and Apportionment, at nine selected points along the west front, and if provided with buildings covering the piers and containing a number of floors, and with modern facilities for lifting and transferring, not only the freight but the cars themselves, and teams and trucks to and from such floors, there can be no question that the railroads' freight business would be handled effectively and cheaply, greatly to the advantage of the City, and largely to the financial advantage of the railroads, by a reduction in the number of leases involved by their present methods.

Such a development is suggested in Fig. 9, in which a five-story building, on a dock 700 ft. by 100 ft., affords space for locating and loading or unloading 150 cars, with ample room for teams and trucks on intermediate floors, all goods being disposed of by chutes extending from the platforms of one floor to those of another.

The cars would be lifted from the car-floats by exterior elevators, and transfer tables would align the cars on the respective floors. Trucks would be elevated to the proper floors, and descend at the opposite end of the building.

Such a use of water-front property eliminates the disadvantageous features of crossing the marginal way and West Street, either at grade or overhead, and concentrates the handling of goods at the points of major movement, creating in a vertical direction the additional amount of space required by the growth of traffic, a process which has been followed in every other business except that under consideration.

The propositions, advanced by Mr. Cresson and the Dock Commissioner, for dealing with the second and immediately pressing problem of the provision of longer piers for the great vessels soon expected in this Port, are equally lacking in purview, and seem to be rather helplessly confined to a single method of solution of the difficulty. This solution takes the form of a proposition to destroy at short notice an institution of long standing, namely, the West Washington Market, thus introducing a complete change in a business of considerable importance, and effecting radical alteration of certain

Mr. Bolton.

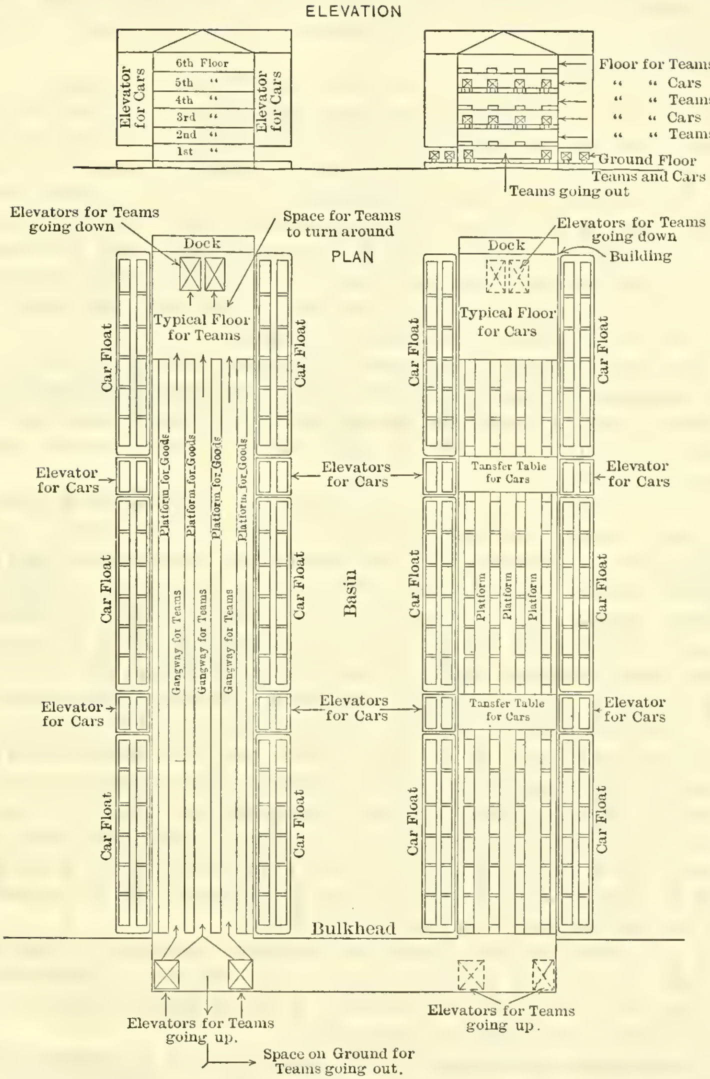


Fig. 9.

Mr. Bolton. methods of business, all for the purpose of securing a single site on which the Dock Commissioner appears to have set his mind as the only possible position for a long dock in the immediate future. The plan also involves a radical re-arrangement of the important West Side marginal street.

The situation which now confronts the City in this matter is a logical result of the stupid and grasping methods by which the invaluable water-front has been allowed to be filled in to such a point that proper space no longer exists for the length of piers which are now, and will in the future be, required; but it will be evident that, if the planning of the future arrangements of the West Side water-front are seriously taken in hand, a more practical and permanent solution can be found than the proposition of the Dock Commissioner, to tear out the market and construct one or two long piers which, at the most, would afford only a temporary relief.

The proper course to be taken is evidently the reconstruction of some considerable portion of the water-front improvements in the form of piers at an angle with the present position, by which piers up to a length of 1 200 or even 1 500 ft. could be provided at several points along the water-front.

Had this course been taken in connection with the West Chelsea dock improvement, the present difficulty would not have arisen. It is an open question whether the handling of these great vessels could not be accomplished better by the provision of a landing stage for passengers, baggage, and express matter, and the reloading and coaling of such vessels at an island dock near Liberty and Ellis Islands, in which these processes could be conducted at a higher rate of speed than is possible with the present dock system, or alongside the street.

It seems strange that vessels, the charter value of which runs from three hundred to many thousands of dollars per diem, should be served so ineffectively under present conditions as to require their presence at a dock for days at a time. Under future conditions, one would expect to see such vessels unloaded, reloaded, coaled, provisioned, and dispatched within not more than 24 hours after their arrival, whereby a much greater use could be made of the investments, both of the vessels and the docking and other facilities provided.

A suggestion for such rapid and intensive operations is outlined in Figs. 10 and 11, a plan and section of a loading and coaling dock provided with means for the transfer of goods directly from and to railroad cars in buildings on both sides.

Finally, this consideration of the subject leads to the enquiry why the present pier system should remain spaced according to the street spacing, whereby vessels are handled only from one side at a time. It would appear to be desirable that a great deal more study and thought be devoted to this whole subject by the Department of Docks, before

Mr. Bolton.

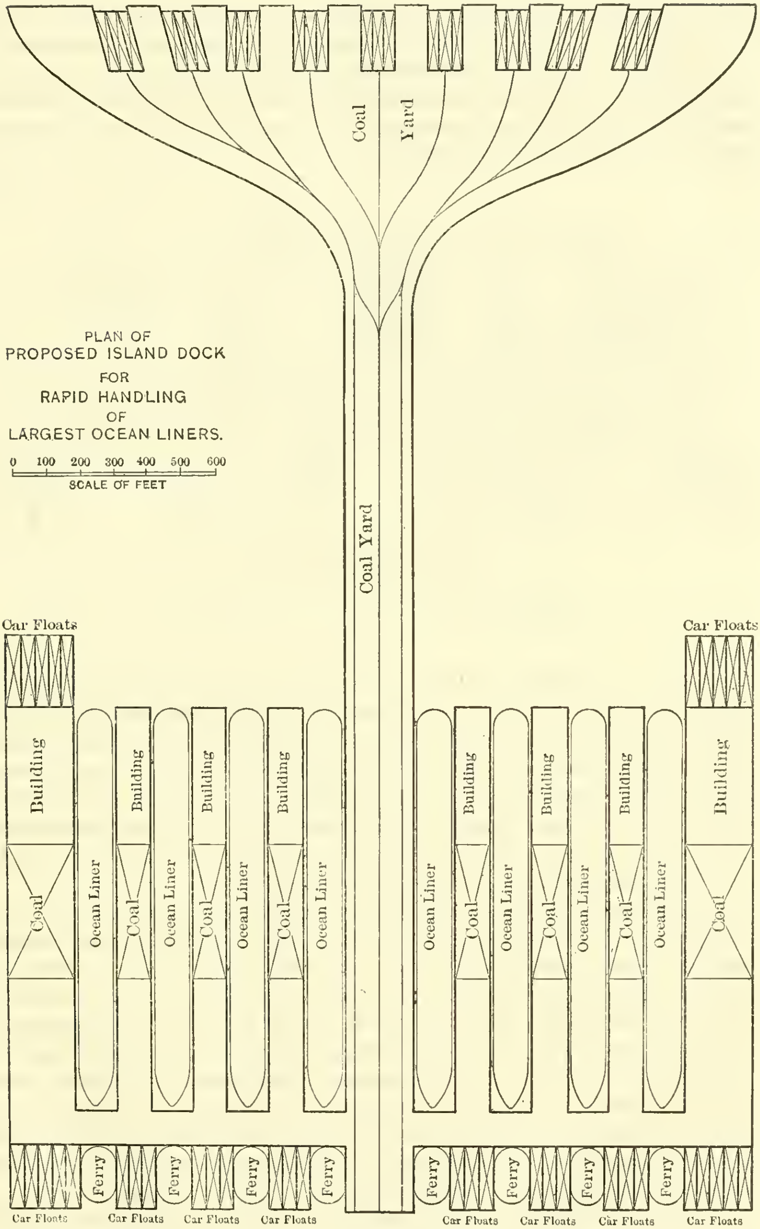
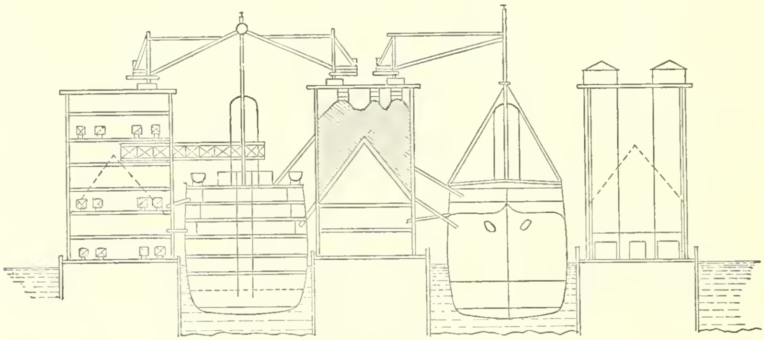


FIG. 10.

Mr. Bolton. attempting to carry into effect measures for a solution of the difficulties which have grown up around the imperfect methods of the past.

It is to be hoped that before any of the remedies proposed by the Dock Commissioner are fastened upon the City, very thorough consideration will be given to methods and means of relief, some of which have been briefly outlined herein.



ELEVATION OF
PROPOSED ISLAND DOCK
FOR
RAPID HANDLING OF LARGEST OCEAN LINERS.

Fig. 11.

Mr. Seaman. Henry B. Seaman, M. Am. Soc. C. E. (by letter).—Mr. Cresson's paper is a very complete presentation of what has been termed the "West Side Problem" of Manhattan. He naturally views the subject from the standpoint of the water-front, which shows, perhaps, its most important commercial aspect. Another consideration, however, is almost as important, namely, the elimination of the steam railroad on the street surface of Eleventh Avenue, and, also, the reduction of the cost of cartage through the streets of the city.

Several years ago, in behalf of the Public Service Commission of the City of New York, the writer took up the question of city cartage, and the results then reached were not very different from those now presented by Mr. Cresson.

The West Side problem is, indeed, one of the most important which the City faces to-day. Rapid transit subways, from a purely engineering standpoint, are practically solved. Their present difficulty seems to be a financial, and, it might be said, a political or personal, rather than a scientific one. but the West Side problem, while including difficulties with vested interests, is, in the main, a question for broad and intricate engineering investigation.

The phase of the subject which the writer had under consideration was the decrease of the cartage congestion through the streets of

Lower Manhattan, and, at the same time, as already mentioned, the elimination of the railroad tracks on Eleventh Avenue. Mr. Seaman.

The amount of cartage through the streets is proportional to the distance hauled, that is, if the distances were reduced one-half, the cartage (except for delays at the terminals) would be decreased proportionately. Furthermore, in order to decrease the congestion and the delays at terminals, the latter must be more widely scattered about the city, rather than concentrated in one locality.

In order to decrease the cartage, goods in bulk should be delivered as near their point of destination as possible; and, to accomplish this, there should be a series of terminals around the belt of the district served. As Lower Manhattan is about 2 miles wide, these terminals might be about 2 miles apart—closer in districts of congested business, and farther apart where business is more sparsely distributed. These distances would probably vary from 1 mile to 4 miles, depending on local conditions. By this arrangement, not only would the haul through the streets be shortened, but, at the same time, the terminal work would be distributed and extreme congestion avoided at these points. The cost of local cartage would be reduced, because of less delays at terminals and shorter hauls, and local expenses would be more nearly comparable with those of other ports.

If this method of distribution were adopted, the next consideration would be how to reach these terminals. A belt line is the natural means; but should it be on the water which surrounds the island, or should it be by elevated railroad?

The waterway has been generally recognized as the cheapest and most elastic means of transportation, particularly as most of the railroads are on the New Jersey shore of the North River. It is a route which has been afforded by Nature, and, in the early days of New York City, before traffic had grown to its present proportions, it was the only practicable means of delivery.

The use of the waterway necessitates either that the terminal be on the river side of the marginal way, or that tracks be laid across the street at grade, because it does not seem practicable to elevate the cars to an overhead crossing within the short distance from the river. The introduction of more grade crossings would not be tolerated, and the river-side terminals now in use, although a natural development, have grown until they have reached such proportions as to cause the present congestion and demand early correction.

It is due to this condition of local distribution that the proposition of an elevated railroad is presented. The harbor has been established so long on its present lines that a complete development of railroad connections, with piers, warehouses, and local terminals, must be one of gradual change, or evolution. The railroad is already needed to relieve congestion at certain points, and also for the purpose of

Mr. Seaman. eliminating the railroad on Eleventh Avenue. If transatlantic vessels continue to increase in size, as they probably will, the oblique pier in the North River will be a necessity, and this, in turn, will facilitate railroad connections.

By the process of elimination, the writer concludes that the elevated railroad is the ultimate solution, but the City should look far enough ahead to avoid temporary expedients. The proposition to construct float-bridges is decidedly of a temporary character. Their use would be cumbersome and restrictive. A tunnel under the North River, with complete railroad connections in New Jersey, would be an essential of the development which the writer believes would be most wise to construct at the outset.

Mr. Gandolfo. J. H. Gandolfo, Assoc. M. Am. Soc. C. E. (by letter).—The problem of handling the steadily growing freight traffic in and about New York City, and of providing adequate docking facilities for the ever-increasing steamship business in this port, should receive careful study, and nothing on a large scale should be done, either by the municipality or by private interests, until the problem has been considered from every point of view and the rights of all concerned so correlated as to work no injustice to any one company or individual.

It is to be regretted that Mr. Cresson did not go into more detail, and give estimates on which arguments could be based. For example, it would be interesting to know the cost of such a terminal railway as he proposes, what returns the City could expect for such an outlay, and whether the income would be sufficient to make it a paying investment and not throw an additional burden of taxation on the people.

Mr. Cresson speaks of "permanent overhead rights" to be acquired by a railroad. It is very doubtful if any such rights can be granted "in perpetuity" by any legislative body, as it is contrary to the fundamental principles of law that any legislature can limit or restrict the powers of any succeeding legislature. In this the writer uses the term "law" as referring to primary principles, not to "law" as interpreted or administered by present-day jurists.

In regard to a municipal elevated railroad, and its control by the City, it is regrettable to have to state that, in New York City, at least, municipal control is a failure. In support of this statement the writer calls attention to the following five cases, taken at random from the City Departments:

(*a*) The present wretched condition of the street pavements throughout the city.

(*b*) The absolute failure of the Street Cleaning Department to keep the streets in anything like a cleanly condition.

(*c*) The complete failure on the part of the Police Department to give that protection to the law-abiding community which the citizens have a right to expect and demand. Mr. Gandolfo.

(*d*) The fact that the water supply system of a great city like New York is laid out, constructed, and controlled, so that the breaking of a single pipe in some sections of the city deprives entire districts of water for hours at a time, and exposes them to all the dangers of a conflagration.

(*e*) The statement by the Park Department itself that certain parks are in such a condition that the lawns must be entirely made over, and that large sums of money must be spent for this work, although this Department was supposed to be taking care of the parks; and the statement of this same Department that the way to take care of the trees and grass plats along a certain avenue is to do away with them entirely and asphalt these areas.

All these examples are in departments under complete and absolute municipal control, and these conditions continue to exist in spite of the fact that large sums of money are appropriated annually for the maintenance and extension of their work.

One often hears the expression, "crowded New York." Certain districts are very much crowded, but it is a fact that there are large areas, close to congested districts, of which very little use is made, and this is the case along lower West Street. With a few exceptions, such as the old and new Whitehall Buildings, 90 West Street, the Central Building, and, farther up-town, a few factories and warehouses (and those who are familiar with this section know how very few these are), the east side of this street is occupied by old and dilapidated buildings only three and four stories high, and, in many cases, of only one and two stories. These buildings are occupied for the most part as beer saloons, small shops catering to sailors and 'longshoremen, horse-shoeing establishments, and things of a similar unimportant nature. Buildings of the same class occupy not only the blocks extending between West and Washington Streets, but also a large percentage of the blocks extending to Greenwich Street, the next street parallel to the river. Thus, there is an area of a full city block, and often of two city blocks, extending back from the river along West Street, of which very little use is made. In regard to relieving the congestion along this thoroughfare, the writer wishes to present the following schemes in outline. The ideas here set forth have to do with the handling and storage of freight and the relief of the congestion along the docks and bulkheads. No attempt is made to go into the matter of providing longer piers, as the writer has not yet made a detailed study of this part of the

Mr. Gandolfo. problem. In discussing this matter, the subject will be divided into two parts:

(*A*) Handling freight to and from steamships.

(*B*) Handling freight to and from railroad cars.

(*A*)—*Handling Freight to and from Steamships.*—Many of the docks along West Street are of primitive construction, consisting of only a shed roof to protect the contents of the pier from the weather. These docks should be replaced gradually by modern two-story structures, and equipped with modern appliances for handling freight. The steamship companies should acquire the property along the east side of West Street and opposite their piers; or, it might be advisable for the City to obtain control of the property opposite such piers as it owns, and lease it with the piers, thus, although controlling the property, permitting its management and use by private interests, and deriving a revenue therefrom. On this property, large warehouses should be erected, and equipped with all modern appliances. The first floors of such buildings should be devoted to platforms for loading and unloading trucks; thus, if an entire block was secured, giving four sides for the arrival and departure of trucks, and as many interior driveways for loading and unloading as a detailed study would show to be advisable. From these platforms, vertical conveyors and platform elevators should extend to the floors above.

The second floors of these buildings should be designed so as to act as a feeding space for the shore ends of conveyors, which should be carried across West Street on light bridges, and extended out on the docks at the level of the second floor. A conveyor of a trolley type, with overhead track and individual carriages, a continuous belt conveyor, or a moving platform, can now be built so as to handle a great variety of miscellaneous freight. The writer has studied conditions on many piers, and finds that in a majority of cases nearly all the freight on a pier can be handled with such a system. The very small percentage which could not be handled in this way could be taken care of on the first floor of the pier, as is done at present. On first thought, one objection to this scheme might be that all goods must be elevated to the second-floor level; but, as a large part of the freight is kept in warehouses, it would simply be a question of bringing it down to the second floor, instead of the first. Further, in loading a large vessel, the freight is now raised to a high level by booms on the dock or the ship and dropped into the hold, and it would be just as easy to sling this freight from the second-floor level and lower it into the hold; and *vice versa,* in unloading vessels. The conveyors for such a system should be reversible, so as to work equally well in either direction.

(*B*)—*Handling Freight to and from Railroad Cars.*—The various railroads should acquire the property along the east side of West Street, and opposite the piers controlled by them, or such property should be controlled by the City, as previously stated. Loading slips for car-floats should be provided at the river ends of such piers. An incline, on which the cars could be handled by a cable and bogie truck, should extend from near this end of the pier to the second-floor level. The tracks would then be carried along the pier at this higher level, and across West Street on bridges. Two, three, or even four, tracks could thus be provided along the dock. On the east side of West Street, warehouses similar to those described under (*A*) should be built, except that the second floors should have loading and unloading platforms to accommodate freight cars. As the tracks come from the docks, they should expand, and by ladder tracks, or any other arrangement of switches and curves, reach the tracks through the warehouses. By a proper layout of these tracks, it would be possible to use the blocks adjacent to those directly opposite the pierheads, and also extend the tracks across Washington Street and use the blocks between Washington and Greenwich Streets. It is probably needless to say that movement on such a railroad should be by electric locomotives. Mr. Gandolfo.

As already stated, the warehouses for the railroads should be similar to those for the steamship companies, the first floors being used for handling freight to and from trucks, and the floors above the second for the storage of general merchandise. By a proper arrangement of elevators and conveyors, the handling of freight within the warehouse could be reduced to a minimum.

The advantages of such arrangements are as follows:

(1) Each steamship line or railroad would control the movement and handling of its own freight, and would not be interfered with in any way by any other interests.

(2) It would relieve the congestion along West Street, as there would be four street façades to every block, where trucks could arrive and depart.

(3) It would entirely relieve the piers and bulkheads from the handling of car freight.

(4) It would obviate the necessity of car-floats standing in the slips all day. As soon as a float was unloaded, it could be loaded again with cars ready to be transferred to the sorting yards in New Jersey.

(5) The first floors of piers thus used for car tracks could be given up to such lighter merchandise as would not warrant the expense of installing such a system of conveyors as described under (*A*).

(6) It would reduce to a minimum expensive hand-trucking; holding trucks idle while waiting their turn at the string piece; early

Mr. Gandolfo. closing day, on account of breaking up the rafts of floats; and many floats standing idle while others are being loaded.

(7) West Street is from 200 to 250 ft. wide along the greater part of its length below 23d Street, part of this space now being used for sheds for the temporary storage of merchandise. The congestion along this street having been relieved by these means, and there being no longer the necessity for this storage space, the slips along this water-front could be lengthened inland from 50 to 100 ft., and still leave ample street space.

As Mr. Cresson states, any scheme of this kind is capable of almost indefinite alteration or extension. For example, the conveyors to the steamship piers, in some cases, might be carried under West Street in subways, provided too much trouble would not be caused by water. It might also be found advisable to carry overhead tracks along West Street for short distances, so as to provide better connection with piers and warehouses.

Mr. Tompkins. E. De V. Tompkins, M. Am. Soc. C. E. (by letter).—The development of the Port of New York presents at this time a subject of peculiar interest, not only to the Engineering Profession, but to those whose business interests are affected by its facilities, and to the many who are handicapped so seriously by the present intolerable congestion of the Lower West Side Manhattan water-front.

Mr. Cresson's paper ably presents the solution of the latter problem, as reached by the engineers of the Dock Department. The following remarks are offered, not in a carping spirit, but in the hope that sincere criticism may induce the author, in closing the discussion, to reply.

It is stated that Greater New York has a water-front of about 450 miles, the entire frontage of Manhattan alone being about 30 miles. Comparatively speaking, the water-front of Manhattan, however, is fully improved with more or less modern piers, and a large percentage of its marginal way is intolerably congested, while the 400-mile water-front of the other four boroughs is practically in its natural state.

The plan described in the paper would be a most excellent scheme for retaining in Manhattan for another twenty-five years all its present industries; but, while of benefit to one borough, would it not retard the development of the city as a whole? Many manufacturers, seeking the advantages of this proposed elevated freight railroad, would flock from other parts of Manhattan to the water-front, and the congestion would soon be even greater than it is at present, on both the land and water sides of the marginal way.

It was stated in the oral discussion of the paper that appliances for the mechanical handling of freight could be readily installed in one of the proposed terminal buildings occupying a block, say, 200 ft. square, which would have a capacity of handling 800 tons per hour, assuming 1 ton to each truck. Installation of machinery of this capacity may be easy of accomplishment, but how could 800 trucks per hour reach the building? These proposed buildings are to be located on the land side of the marginal way. Probably all these trucks would enter on one side of the building and leave on the opposite side. This would necessitate the passage of 1 truck through the approach street every 4 sec. Even if the trucks could enter on two sides and leave on two sides of the building, a rate of 1 truck every 8 sec. would result, which is far from practicable. Mr. Tompkins.

If the scheme of the Dock Department were to be carried into effect, and used by all the railroads, the City of New York would then consist of four boroughs practically no more developed than they are to-day, and the Borough of Manhattan, indescribably confused and congested. Here would be huddled together office buildings, factories, hotels, and a great rail and water terminal for freight as well as for passengers.

Under the present congested conditions, industries now operating in Manhattan will soon be forced to seek accommodations in other parts of the Port, and the time is ripe for the development of great industrial centers along the 400 miles of unimproved water-front. Many ideal locations are to be found here, suitable for such terminals, which could have several railroad connections to the very doors of hundreds of factory buildings, with ample space available for all sorts of industries. Rents would be cheaper than in Manhattan, both for manufacturer and employee, and the latter could live in more open and healthier surroundings. The congestion along Manhattan's water-front, therefore, might best be relieved by offering such attractions at other parts of the Port.

If, then, as the time is ripe, every inducement be now made to encourage the private and municipal development of great industrial centers uniformly over the four boroughs, as best suited to the conditions of each, the City of New York will consist of the Borough of Manhattan, the oldest and most centrally located, the executive center of the City, built up with office buildings, residences, hotels, and stores, and the passenger terminal both of railroads and steamships, the four other boroughs being the industrial district of the great City and the terminals for all freight, whether by rail or water.

The author, in remarking on the natural advantages of the Port of New York, does not mention its second great entrance, namely, Long Island Sound. This oversight is quite common, probably due to the general habit of considering Manhattan as the whole of New York City. As the present obstructions to navigation in the East River limit the use of the Sound as an entrance to Manhattan, the ad-

Mr. Tompkins. vantages of its use to vast water-front areas in the Boroughs of Queens and the Bronx have been, perhaps, unappreciated. It is not generally known that to-day vessels, even of the *Oceanic* size, can safely navigate the Sound about as far as 150th Street and the East River at any stage of the tide. With the completion of the $37 000 000 project now before Congress, for the removal of the obstructions in the East River, all boroughs of the city will benefit by this second entrance to the Port.

While the writer agrees with Mr. Cresson that all steamship passenger traffic to New York should be handled in the Borough of Manhattan, and that piers of necessary length should be permitted to accommodate it, he does not believe that the immense volume of freight should be handled there, but that this should be diverted to other parts of the harbor where there are extensive unimproved areas for industrial development under more suitable conditions than obtain in Manhattan.

The most serious congestion is caused by the trucking, and it is also the most expensive item in the handling of freight. This is due largely to the retention, at this late day, of that almost obsolete means of transportation, the horse; and, is not the existing method of shipping, which requires each shipper to hire a truck to haul his freight from the factory practically direct to the freight car, almost as absurd as it would be to require each passenger to hire a truckman to haul his trunk from his house to the baggage car of his particular train? Were this latter method of handling passengers' baggage exclusively followed, intolerable congestion and expense would certainly result.

The rapid increase in the size of modern steamships has been most remarkable, but their development cannot be compared with that of the commercial motor-truck. In this latter lies in part the solution of the freight transportation problem in Manhattan. The freight of this borough should comprise only foodstuffs and materials for the personal needs of its inhabitants. Such direct rail connection as may be necessary for this can be provided by an elevated freight railroad built for the exclusive use of the New York Central Railroad at its own expense. To compete with this, other railroads would soon build, at most convenient locations throughout Manhattan, terminal buildings for receiving and distributing such freight. The railroads would transport the freight between these buildings and their railroad yards by their own motor-trucks. The upper floors of these buildings could be leased for light industrial purposes, and revenue could be derived therefrom. While with horse-drawn trucks it is desirable to run cars within, say, 2 or 3 miles of the destination of the freight, with the motor-truck 15 or 20 miles will be covered in less time with three times the load.

It may be readily seen that a large percentage of freight may be handled without passing through these terminal buildings at all, but can be delivered by the railroads directly at the door of the consignee. By using the motor-truck for this purpose, no costly subways, elevated railroads, in fact, no expense at all would be incurred beyond the equipment. Certain longitudinal streets at least should be allotted for traffic of this particular class, though not necessarily restricted to it. These streets should be suitably paved, and sane speed regulations should be adopted. For this proposed method of transportation between the railroad yard and the distant distributing points in Manhattan, a train will be made up (for night work at least) consisting of a single motor-truck (with its operator and mechanician) and three or four trailers. Mr. Tompkins.

The body of each motor-truck and trailer, with its incoming package freight, can be readily lifted from the chassis at the terminal building. At the latter the shipper's freight has been classified and loaded into empty truck bodies, and one of these will immediately replace the loaded body just delivered. The motor-truck will take this to the railroad yard, where the body will again be shifted with several others from the motor chassis to a skeleton railroad car, and when this car reaches its destination, the body will again be shifted to the railroad's motor-truck in this town and thus distributed without the package freight having been handled since it was classified at the terminal building in Manhattan. In the future, therefore, the railroads would establish free "motorage" as well as free "lighterage" limits.

Were this method adopted, the proposed North River Bridge would immediately become of inestimable value to the railroads now having their freight termini on the New Jersey side of the river, as well as for vehicular traffic in general, for interstate surface railroads, and for an interstate highway and recreation place for pedestrians.

To conclude, it is suggested:

First.—To locate on the undeveloped 400 miles of water-front of the Port long piers, supplied with railroad tracks for through freight, and equipped with the best possible mechanical handling appliances, also modern factory buildings and warehouses immediately adjacent thereto, thus attracting manufacturers and shipping away from Manhattan.

Second.—The New York Central will provide one all-rail route to the lower part of Manhattan.

Third.—To install a well-devised system of motor-trucks to take care of its share of the greatly reduced traffic that would result under these conditions.

It is well known that any and every plan which tends toward the improvement of the Port of New York will have the hearty cooperation of the present administration of the Dock Department. It

Mr. Tompkins.

is desirable that there should be a thorough discussion of Mr. Cresson's paper and constructive criticism made of the same. Engineers, in particular, should devote much thought to the subject, and should aid the present Commissioner in his efforts to create an overwhelming public demand for immediate action.

Mr. Robinson.

A. W. ROBINSON, M. AM. SOC. C. E. (by letter).—To provide adequately for the expansion of the Port of New York is a problem which no one man can solve, and it is by hearing all the interests involved and inviting discussion from engineers that those charged with this work can obtain data and suggestions from which finally to evolve an acceptable plan.

The mixture of the various kinds of traffic in large cities, which in early days presented no difficulty, has with growth produced conditions which are wasteful and illogical. Follow the course of a shipment of goods arriving in New York until it reaches the consumer or retailer. After being extricated from the chaos at the ship's side, it is carted by horses through the streets over stone pavements and dumped on the sidewalk in front of the wholesale merchant's premises, being obstructed and obstructing others at every step. The operation thus far has taken as long and cost as much as the sea journey of 3 000 miles. After a more or less prolonged stay, during which it accumulates more delay and added cost, it is shipped out by a similar process to the consumer or retailer. The trucking in city streets would be greatly reduced if only freight for consumption in the city were handled, and all through freight and traffic of wholesalers destined for points outside were handled in some other way. The horse-trucks and rough stone pavements are also archaic, and motor-driven trucks on smooth and level pavements built specially for them would be far more efficient.

As Mr. Cresson has pointed out, the large steamships must be provided for in a central position. The rapid and cheap handling of ingoing and outgoing freight to these vessels is essential, and it should be kept off the street, except for local requirements.

Most schemes which have been proposed contemplate the construction of largely increased railway facilities on the dock front, and the continuance of the present system of car-floats, or a further development of tunnel systems. For steamship freight in transit, both ingoing and outgoing, the writer would suggest that the car-float system be expanded so as to constitute floating warehouses into which cars could be run. Such floating warehouses should have ample floor space on two decks and complete mechanical freight-handling appliances. They would perform all the functions of fixed sheds on the pier, but be carried on steel hulls instead of fixed foundations, and have the advantage of making a connecting link between the ship and any railroad with the least delay and expense. On the arrival of a vessel at her pier, one of these floating warehouses would go alongside, and the receiving and discharging capacity of the ship would be doubled. The lower deck could carry sixteen or more box cars in addition to large floor space level with the floors of the cars, and have complete mechanical loading and unloading appliances. The loading and unloading of these cars could be done while the vessel was changing position, so that on arrival at a railway the cars would be ready to proceed as from the present car-float. Mr. Robinson.

As most of the large steamship piers are now without direct railway connections, and have no means of getting any except by further congestion on the landward side, it would seem worthy of consideration to expand the car-float system in some such way.

It would seem reasonable to avoid bringing into the city any freight which is not destined for consumption there and to minimize trucking by the establishment of large terminal buildings in an attractive locality, for occupancy of business firms as tenants, and supplied with shipping facilities without trucking.

The development of the outlying portions of the harbor is a vast problem almost untouched, and, as Mr. Cresson says, offers a wealth of opportunity. It is to the development of these regions that New York must look to provide facilities for commerce other than local. These outlying spaces will immediately become much more valuable when quick passenger communication is provided by electrically operated tunnels, with Lower New York as the nerve center.

The vast areas of the Hackensack Meadows, the shores of Newark Bay, the west side of the Upper Bay opposite Bayonne, the lagoons of Rockaway, and other places offer opportunities for the creation of many square miles of real estate at a nominal cost. There has been some filling in of the Hackensack Meadows in a desultory way and with slow and costly methods, and what is urgently needed now, before any more work is done, is a comprehensive and authoritative plan covering all these districts, to provide for all needs and to represent and conserve all interests involved. In this way all work should be a part of a systematic plan, and not have to be undone later.

It may be of interest to note what is being done, in the way of harbor improvements and land reclamation, at Bombay. The Port Trust of Bombay is carrying out extensions which include the conversion of tidal flats into a large inner basin, at the same time, the adjoining land is being reclaimed with the spoil. For this purpose two powerful hydraulic dredges of special design have been constructed, of 3 500 h.p. each and capable of dredging 25 000 cu. yd. of soil per day and delivering it at a distance of 4 000 ft. These dredges are fitted with powerful cutters and suction apparatus designed by the writer, and can cut their way into a solid bank, making a channel

Mr. Robinson. 500 ft. wide and 35 ft. deep at one time. The cutter is similar to a gigantic milling tool with steel blades, and cuts a section of from 60 to 70 sq. ft. at a rate of from 30 to 60 ft. per min., according to the hardness of the soil.

There are no dredges on the Atlantic Coast at the present time at all competent to deal with land reclamation works of any magnitude. The unit cost at which such work has been done in a small way heretofore would be prohibitory, but with modern tools of large capacity, adapted to the conditions, these large projects become much more reasonable, and indeed are rendered possible where before they would have been impossible.

Hydraulic shore discharge work for land reclamation has usually been done with small dredges having discharge pipes not more than 20 in. in diameter, so that they can be readily handled and shifted by manual labor. As the pipes in large modern dredges are not handled by manual labor, they can be made of large size, say, from 42 to 48 in. in diameter. The tidal rise can be utilized to lift and move the pipes to a new location, and the land can be built out from shore by a suitable terminal pontoon carrying a long overhang. A terminal pontoon of this sort, with an overhang of 220 ft., was used by the writer in making shore deposit on the Upper Nile. In this case the advance was rapid and continuous; there was no manual labor and no delay due to the shifting of pipes.

These brief references will serve to show that the capacity of modern tools for dredging and land reclamation has an important bearing on the development of the outlying regions of New York Harbor; and, in order to get the best results, the improvements should be designed in conformity with the capabilities of these modern tools. For instance, advantage should be taken of present depth of water, as far as possible, and favorable material should be chosen for the maximum excavation. The distance from cut to fill should not exceed 4 000 ft., for economy at one handling. Under favorable conditions, one of these dredges will reclaim 2 acres per day with a fill of 6 ft.

Mr. Thomson puts forward a bold plan for the reconstruction of the harbor, and some of his remarks are very pertinent. His criticism, that the proposed railway belt line would not relieve congestion, but rather promote it, appears to be well taken. His plan of extending the city 4 miles into the bay, while it would create much valuable real estate, seems open to serious objections. The water passage around the Battery is one of the most thickly traveled in the world, and the navigation interests would never permit it to be closed. Furthermore, the bottle-shaped extension would cause a density of traffic at the neck of the bottle which would be hard to overcome. Mr. Thomson has also located much of his land fill in from 60 to 70 ft. of water, in the main ship channel, which it would be a pity, as well as wasteful, to spoil. His "New Governor's Island" is also located partly in the old ship channel, which has been dredged at great cost. It is easy to draw lines and create new continents and islands on the map, but, nevertheless, the subject is worthy of the most careful study. Without doubt the present contours of these outlying districts could be materially changed for the better, and the cost would be but a small proportion of the increased value, if systematically carried out with modern tools. Mr. Robinson.

AUGUSTUS SMITH, M. AM. SOC. C. E. (by letter).—This paper is a most commendable presentation of a plan for developing the west water-front of Manhattan, and is well worth the serious thought and criticism of every one interested in the welfare of New York, whether or not he be an engineer. It is particularly gratifying to see the interest in the question displayed by the members of this Society, and the discussion cannot but result in a material contribution to the proper solution of the problem. Mr. Smith.

The writer does not agree with Mr. Cresson in advocating the construction of an elevated distributing railway system on West Street, but desires to record his hearty endorsement of Mr. Harrison's discussion. On one point only does the writer suggest a possible amendment to Mr. Harrison's plan. After breaking bulk in New Jersey, freight for Manhattan might be distributed by motor-trucks and trailers instead of by light narrow-gauge flat cars. The tunnels could be equipped with means for hauling through a broken-down truck, or cable or electric haulage could be relied on for all tunnel movements.

B. F. CRESSON, JR., M. AM. SOC. C. E. (by letter).—It affords the writer considerable gratification to have obtained such a comprehensive discussion, but, from the fact that the alternative suggestions are few and usually unimportant, he is led to believe that the principle established in the paper is fundamentally correct. Mr. Cresson.

One criticism, which appears to be somewhat general, is that the other parts of the harbor have not been duly considered; studies have been made, however, for the organization of terminals in various parts of the harbor, indicating the best uses for each, together with a proper consideration of the relation of each to the others, and, in these plans, the New Jersey side of the Hudson River, Newark Bay, and all other available water-fronts have been considered. The paper, however, in order to be as brief and direct as possible, deals with only one of these subjects.

Mr. Hoag's discussion is interesting because it describes the development of the City's water-front from the early days, and calls attention to the fact that, while it has always been possible to establish piers in Lower Manhattan, on water-front property not otherwise needed, conditions have now changed, and there is no room for further expansion except through better organization.

Mr. Cresson. Mr. Harding's discussion is based on a careful study of the situation with the object of a more economical handling of freight by mechanical means. There is no doubt that the present method of handling package freight in this section of the water-front is very expensive, and in many instances will be replaced ultimately by mechanical devices. It is difficult to prophesy in just what form this change will take place, but, as stated in the paper, Mr. Harding, working with the Department, is making a very careful study and investigation of this exceedingly important matter.

As stated by Mr. Harding, the plan of breaking bulk appears to be questionable. If it is possible to load cars for their destination and prevent breaking bulk, that will be the most economical way, and it is believed that this may be accomplished to a large extent by a proper organization.

It is with great satisfaction that the writer notes the approval of the general principles of the plan by Mr. James Forgie. The firm of which Mr. Forgie is a member has successfully planned and engineered the tunnels under the Hudson River, and has solved the problem of connecting New Jersey with Manhattan for passenger service. Its approval of the suggestions for the solution of the freight problem should have great weight.

Mr. Forgie states that he regrets the introduction of a compromise in the shape of transfer bridges to be constructed in the neighborhood of 30th and 40th Streets. It is believed that, in the early stages of the development of this plan, and even after the tunnels have been constructed, a certain amount of car-float business will still be carried on, and, while it is apparently an intermediary stage, it will no doubt be availed of for many years.

Mr. Harrison's argument appears to present the harbor situation from the New Jersey point of view. In the paper, the writer did not attempt to treat the Port as a whole, but merely presented the situation on the Lower West Side of the Manhattan water-front, and, in dealing with this, it should be remembered that its relation to other parts of the Harbor, not only in New York, but also in New Jersey, was carefully considered.

Mr. Harrison calls attention to the fact that the Port of New York, taken as a whole, should include, not only New York, but also the New Jersey section, and, as there is no doubt that this is entirely proper, it has been thus considered in formulating the plans of the Dock Department; but, if it is thus considered, it is a question as to whether free lighterage should not be extended to all parts of the Harbor, which is objected to by Mr. Harrison.

New York itself has developed faster than New Jersey, principally because it has been largely under the control of a single governing body, whereas the control of the New Jersey water-front is split up among small municipalities to such an extent that a general plan has not been adopted, and probably cannot be adopted until some parent commission shall administer the entire New Jersey water-front of the Port of New York. Mr. Cresson.

It is not the intention to concentrate further the business and population in Manhattan by the plan outlined. The business exists in Manhattan at the present time, and the method of carrying on the railroad business with the City, which is the difficult problem, is to be done in the section of the City back from the water-front which is not now intensively used, in order that the water-front may be applied to its proper use. The thought and effort of the Dock Department, in all its projects, have been to provide plans whereby business and manufacturing may be directed away from the congested Manhattan district and to the outlying boroughs.

It is not the purpose of this plan to run cars on the piers in Manhattan, as Mr. Harrison appears to believe, and it has been one of the most difficult things to make it understood that this is not the plan, although it has been most clearly stated many times. The proposed extension of the pierhead line, and comments on the width of the stream are interesting, and a certain extension of this line is believed to be very desirable.

Mr. Harrison's statement that 1 000-ft. piers can be constructed north of Castle Point on the New Jersey shore is entirely true; they can be constructed there, and they can also be constructed in many other parts of the harbor. The fact that one of the lines now docking in New Jersey desires to be provided with accommodations in Manhattan is sufficient evidence that Manhattan is the most convenient place for them.

It is not easy to make statements relative to the cost of operating such a railroad as is planned, but, from very careful study and investigation, it is believed that this scheme can be operated successfully, both from commercial and economic standpoints.

Mr. Harrison's argument relative to small-sized tunnels and small-sized cars brings up a plan which was presented some years ago, but never adopted. Some of the objections to the proposed use of small-sized tunnels and small cars are as follows: The compulsory breaking of bulk and consequent necessary rehandling would add greatly to the cost of operation. A subway along the westerly marginal water-front would probably interfere with the sewers, and would necessitate reconstruction on a different level. The cost of constructing subways along the westerly marginal way would be great. In making connections to buildings in the comparatively narrow streets back from the water-front, it would necessitate rearranging the vaults and other sub-surface structures. The difficulties and dangers of operating a freight subway are noted in the paper. Small-sized tunnels with

Mr. Cresson. corresponding cars might make it impossible to handle a considerable quantity of freight shipped in large cases. There is no doubt as to the truth of Mr. Harrison's statement relative to the present method of transferring freight to and from vessels on the Manhattan water-front as being the best, cheapest, and most elastic, and there is no intention whatever of changing the method of handling this business. The plan is to handle the business which the railroads do with the City itself, and not with the ships.

Mr. Higgins calls attention to the fact that all the railroads connecting with the West have their terminals on the New Jersey shore, and that it would be easy to tranship freight directly to ships on that shore.

This, of course, is true, but the stated advantage of it does not seem to be recognized by many of the shipping interests. It is probably not desirable to make regular sailings from the terminal of one road, but if joint railroad terminals could be established in New Jersey for ships, it would, no doubt, be attractive to a certain class of vessels carrying bulk freight. There are facilities for transfer between rail carriers and ships on the New Jersey side which are seldom used. It seems necessary again to state that the plan outlined in the paper does not concern the steamship freight.

Mr. Lewinson's objections, from a financial standpoint, are not convincing. The City of New York has a large dock fund out of which the plans outlined may be financed, and, by leasing these facilities on a self-sustaining basis, the funds necessary to construct them could again be authorized for further dock improvements. There is no suggestion in the paper that any land will be condemned.

Mr. Thomson, in his criticism, refers to additional congestion by bunching the bulk of the freight in a few terminals. Again, it seems necessary to call attention to the fact that the steamship freight will be handled on the water-front and that the railroad freight may be handled in as many terminals as are needed on the easterly side of West Street, and located where they are most convenient and accessible.

His argument that freight from Philadelphia to Lower Manhattan must go miles out of the direct route is not a serious defect, for, when freight is on the rails and *en route,* an additional haul of a few miles is negligible, as the cost to railroads for handling freight per ton-mile is a small fraction of a cent.

The objection to the number of tracks is not well taken, because the capacity of the railroad, as planned, will be very much greater than will accommodate the railroad freight which now comes to Manhattan.

Train schedules and train movements have been worked out, not only with the transfer bridges, but also with tunnel connections, and, as it has been considered that the freight should leave the terminals in the same volume and at the same times as it does now, the installation suggested appears to be liberal. These conditions might never be realized in actual service, as the sending out and bringing in of freight can be spread out much more over the 24 hours than under the present method of operating cars on car-floats. Mr. Cresson.

As to Mr. Thomson's alternative plan, it should be observed that it is by no means novel, having been brought up from time to time for the last twenty years. The project of joining Manhattan to Governor's Island has been presented for approval to the Army Engineers, and has been denied. The scheme of the late H. Arnold Ruge, of the Dock Department of New York, entitled "Southern Extension of New York" was published, with a plan, in the *New York Times* on October 1st, 1893, and again in the *New York Press* on June 3d, 1894. It was presented by him before the Municipal Engineers on November 27th, 1897, and a bill was introduced in the Legislature (Senate Bill Introductory No. 969) on April 12th, 1909, looking to the investigation of this matter; therefore, it is not something which has not had very thorough consideration in the past.

Some of the important objections appear to be: First, the doing away with the water route around the Battery as a connection between New Jersey, the North River, and the East River. It is believed that this would never be permitted by the interests using the Harbor, for if an additional barrier is made by further separating the East and North Rivers, by, perhaps, 11 miles, commerce will surely suffer greatly. It will never be possible to regulate business in the East and North Rivers so that it will be unnecessary to have direct and convenient connection by water. This passage around the Battery is one of the most intensively used portions of the Harbor, and its value cannot be over-estimated.

Second, at the present time, there is a considerable flow of water from the East River into the North River and *vice versa,* because of the rather narrow passage of the Buttermilk Channel. If the passage between the Battery and Governor's Island should be closed, the currents in the Buttermilk Channel would be worse than they are now at Hell Gate, and the Government is anticipating the expenditure of enormous sums of money to open up the Bronx Kills and Little Hell Gate, in order to reduce the currents in Hell Gate.

Third, the Upper Bay is where the sewage-polluted waters of the rivers meet the clean water from the ocean, and it is there that oxidization takes place. The condition at the present time is that the danger point is being approached, where the sea water will be insufficient to oxidize the sewage fully, and if this Upper Bay is filled in to any extent, the conditions may become such as to render the harbor

Mr. Cresson. waters highly dangerous to the health of the community. There are other objections which might be brought out, but the foregoing are considered the most important.

As to additions to Staten Island, this seems wholly unnecessary as practically none of its present water-front is developed for commerce, and there is room to construct piers and terminals all around its shores. Therefore, why add more land until the present is under some form of development? It should be borne in mind that, of New York's great water-front, a very small proportion is developed, but, by proper organization, it may be developed and made to accommodate an almost unlimited commerce without filling in any of the harbor waters.

As to a navy yard off Sandy Hook, the writer can scarcely imagine a worse location from the point of view of the naval authorities. It would be in an unprotected location, would require very expensive breakwaters, and would be in a most exposed position for attack by a hostile fleet. One of the advantages in having the Navy Yard in its present location or at some other portion of the harbor inside of the Narrows, is that it would be difficult to bottle up any portion of the fleet, there being egress by two channels, that is, by the East River through Long Island Sound and directly through the Narrows to the Atlantic.

Mr. Moore was appointed by the Board of Estimate as one of the consulting engineers to consider the West Side matters with a view to solving the problem as to the proper disposition of the surface tracks of the New York Central Railroad. As stated in his discussion, he received a great deal of information from the railroads, but this has not been given to the Dock Department. It is a source of satisfaction to know that he has reached practically the same conclusion, as to the method of solving this problem, as that of the Dock Department.

Mr. Bolton does not appear to have a very clear understanding of the problem or the proposed solution. His statement about "hauling a freight car from the vast yard in Brooklyn, proposed by the Dock Commissioner, down under the East River, up again to Manhattan along the west front, to some point to be transferred into the terminal, etc.," is an example of this. There is nothing of this sort proposed in the plan, and Mr. Bolton's attention is directed to the map which shows the location of the proposed yard and approach and its connection with the elevated railroad.

If the plans of the Dock Department are carried out, it would still be possible to construct an elevated railroad for passenger service along the water-front.

The suggestion of multiple-story piers is by no means novel, but, as outlined, it would be inoperative. Further than this, the river bottom in the North River is soft in very many places and can scarcely maintain the present load without subsidence. It is not a question of providing more piles, for, if the bottom is loaded beyond a certain point, displacement of the silt will occur. In addition to this, by concentrating additional business on the water-front side of West Street, the drayage congestion would necessarily also be increased, and one of the great objects of the plan would be defeated. Mr. Cresson.

The use of the coaling apparatus is not clear, but, if it were installed as suggested, it would not be possible to load freight at the same time as the ship is coaling, as is done now.

The method of providing longer piers is carefully discussed in the paper. In the later installations of longer piers in Lower Manhattan, their spacing has not been made to conform to the street ends.

It is interesting and valuable to note that the conclusions reached some time ago by Mr. Seaman, as Chief Engineer of the Public Service Commission, as to the necessity for an elevated railroad should agree so closely with the conclusions arrived at by the Dock Department.

Mr. Gandolfo's discussion as to the relation of this West Side matter to the development of other parts of the Harbor has already been discussed.

The writer has been criticized frequently for not having presented figures to show the relative cost of handling freight by the methods proposed as compared with the present methods. It is difficult to find out what it costs the railroads at the present time to bring their freight from New Jersey over to Manhattan, and it is difficult to determine what the cost of operation would be under the methods planned. It has been stated, by men supposed to have authoritative information on the subject, that the present cost to the railroads of delivering freight to Manhattan from New Jersey amounts to between $1 and $1.50 a ton. This includes placing the cars on car-floats in New Jersey, floating them across the river, unloading on the piers, and overhead charges connected with the installation.

In the operation of a tunnel connection from the yard in New Jersey to the elevated railroad in Manhattan, if it is assumed that 10% of the investment should be the overhead charge, which would include interest, amortization, depreciation, and maintenance; if it is assumed, further, that all the railroad tonnage from the New Jersey roads to the City itself, which is now handled by car-floats, would come over this line, and if there is added to these overhead charges the cost of operating this tonnage, coming in railroad cars loaded to about the same extent as they are at present, the cost, to bring the freight in the railroad cars from the yard in New Jersey to the doors of the terminals in Manhattan, would not be more than 40 cents a ton. Adding to this a charge for handling the freight in the terminals and an overhead charge for the terminals themselves (which would be

Mr. Cresson. largely, if not entirely, offset by rental of overhead space), it would still appear that the enterprise should be attractive from an economic point of view.

Relative to the argument about municipal control, it should be remembered that municipal control does not necessarily mean municipal operation, and, in view of the Dock Department's experience in operating the municipal ferries, it is not ready to recommend municipal operation for this elevated railroad and the freight terminals, believing that a private operating company, under City control to a certain degree, should act as the City's agent in this matter.

In reference to the handling of freight to the steamships, the suggestion of larger piers and double-deck sheds is along the lines which the Department is following in its new developments. The addition of warehouses on the easterly side of West Street, and connected with the piers by mechanical conveyors, has been considered, and it is thought that something of this sort might be worked out to advantage. The east side of West Street must be connected with the water-front in some way, and the manner suggested by the Dock Department has been through the medium of an elevated railroad.

Relative to handling freight to and from railroad cars, the plan suggested has been considered, and it is not thought that the cars can be elevated to the piers with sufficient ease and elasticity of movement to have anything like the capacity required.

The argument that each steamship and railroad should control the movement of and handle its own freight, and not be interfered with by other interests, would mean that the water-front would be in very much worse condition than it is at the present time. Each railroad has now established, on the average, three terminals on the North River water-front, at different localities, in order to serve particular districts. If provision is made for the same thing on the easterly side of West Street, practically the entire water-front would be taken up by the railroads, and the water-front itself would have to be permanently surrendered to the interests constructing and owning the terminal buildings, as it would be a necessary part of the installation.

The City has been acquiring the water-front at great expense, and it should be the steadfast policy to keep control of it so that it may be used in the best interests of the Port. It should be noted that Mr. Gandolfo objects to permanent rights being given.

Relative to the statement that it would obviate car-floats standing in the slips all day if, as soon as a float was unloaded, it should be loaded again ready for transfer to the shipping yards in New Jersey; this is not the method which the railroads use in loading freight in Manhattan. A large amount of classification is now done in Manhattan, and the reason that the cars stand in the slips all day is to provide for this classification and prevent considerable rehandling in the assorting yards in New Jersey. Mr. Cresson.

The suggestion that the slips can be lengthened from 50 to 100 ft. by the narrowing up of West Street should not be considered. This street has been widened at a very great expense, it is not too wide at the present time, and nothing should be done to decrease it.

Mr. E. De V. Tompkins' discussion appears to be based on the desirability of providing terminals for manufacturing in the outlying districts and against the establishment of additional facilities for manufacturing on the Island of Manhattan. It has previously been stated that it should be the policy of the City to divert manufacturing industries from the central borough.

Mr. Tompkins states: "Many manufacturers, seeking the advantage of this proposed elevated freight railroad, would flock from other parts of Manhattan to the water-front, and the congestion would soon be even greater than it is at present, on both the land and water sides of the marginal way." Then he states that this elevated railroad should be built for the exclusive use of only one of the carriers. Just how much difference it would make to the manufacturer if this road is built for only one road or for all the roads is not easy to estimate, and it certainly should not be the policy of the City to give exclusive facilities to any corporation.

The use of motor-trucks is becoming more and more extensive, but the additional handling of freight which would be necessary, under Mr. Tompkins' proposal, by the other railroads in inland terminals would probably add greatly to their expenses, in doing business, as compared with those of the New York Central.

Every effort should be made to encourage industrial development in outlying boroughs, in order that the congestion in Manhattan may be relieved.

Mr. Robinson's discussion as to railroad and steamship freight is interesting, but inasmuch as this scheme is for the handling of the railroad business with the City and does not involve the steamship business, which is now carried on economically, it is not pertinent to the question. Cars on car-floats are seldom placed alongside the ships, except in the unloading of bananas.

The object of Mr. Smith's suggestion, that instead of using light, narrow-gauge, flat cars in the tunnel, motor-trucks and trailers be used, is not easy to understand. It would be difficult to ventilate a long tunnel if gasoline motors should be used.

Mr. Calvin Tomkins has dwelt on the subject of the general organization of the Harbor and its relation to the paper, and it should be remembered that the plan outlined is a part of the general scheme which Commissioner Tomkins has worked out for the development of the whole Port.

Mr. Cresson.

At the present time, the matter which is probably of the greatest importance to the City of New York is the prompt and proper solution of the water-front problem of the Lower West Side of Manhattan, and the writer believes that the method suggested in the paper is the only one by which relief can be obtained.

In conclusion, the writer wishes to express his appreciation of the discussion which has been brought out, but, nevertheless, he does not feel that it has been such as to warrant any modification in the plan proposed in the paper.

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TRANSACTIONS

Paper No. 1223

SPECIFICATIONS FOR THE DESIGN OF BRIDGES AND SUBWAYS.

By Henry B. Seaman, M. Am. Soc. C. E.

With Discussion by Messrs. S. W. Bowen, Victor H. Cochrane, Albert I. Frye, F. W. Gardiner, Almon H. Fuller, S. M. Swaab, J. B. French, Charles E. Conover, and Henry B. Seaman.

In connection with his duties as Consulting Engineer to the Department of Bridges, New York City, in 1906, the writer found occasion to take up the revision of the specifications for the design of bridges, in order that a single specification might cover the general work of the Department. The bridges of the city varied in length of span from 16 to 1 600 ft., and were classed as either "long-span" or "short-span," there being none of such intermediate length as would question the class to which it belonged.

It had been the practice of the Department, as it had been the general practice elsewhere, to use a special specification for each bridge, conforming to the particular conditions of span and loading. For short-span bridges, where the live load was heavy and frequent, the allowable unit strains on the material were comparatively small, while on the longer spans, where the live load was more diffused and less frequent, the allowable strains were correspondingly increased. In spans of extreme length, where live-load congestion rarely occurs and is slowly applied, the allowable live-load strains may equal those permitted for dead load.

Although the short-span bridges of the Department were at that time designed somewhat promiscuously—each designer selecting his own specification—the long spans had received more careful attention, and were designed under the recommendations of a Board of Engineers, composed of George S. Morison, Henry W. Hodge, Mansfield Merriman, Theodore Cooper, and C. C. Schneider, Members, Am. Soc. C. E. The recommendations provided for two different conditions of loading:

1st. An assumed, heavy, "congested" load, which would rarely, if ever, occur. For this loading the allowable unit strains were high, being the same as those allowed for dead load.

2d. An assumed "working" load, lighter and of more frequent occurrence than the "congested" load. For this loading the allowable strains were reduced one-half.

This double specification involved a double set of calculations throughout, and, as the computation of each set required some 6 months or more, the necessary time for the preliminary work alone was a serious matter.

Confronted by these conditions, the work of revising the specifications was undertaken. The designers of the Department were all called into consultation at various times, and, after several months of deliberation, a specification was outlined which was considered practicable. This was about ready for adoption when the writer left the Department of Bridges to enter the employ of the Public Service Commission for the First District of the State of New York, as Chief Engineer.

With the work of the Public Service Commission, this general specification was again taken up, for the purpose of extending its application to the broader field including subway construction. It was found suitable for that work, with but slight modification, and was finally adopted on January 1st, 1909, practically in the form here presented. For completeness of record, the items which are especially applicable to bridge construction are retained in this paper.

In taking up the theoretical consideration of this general specification, there seemed to be no sharp line of demarcation between "long-span" and "short-span" bridges. The two extremes appeared to merge gradually as the lengths of the spans changed. There was a fundamental underlying condition, however, which made a broad specification natural and fitting. This underlying condition was the fixed power of resistance of the materials of construction.

It seemed evident that, in order to produce a uniform specification, such elements as were constant in nature should remain constant in the specification, while those which varied under differing conditions, should be made to vary, with those conditions, in the specifications. The strength of materials—or their power of resistance—is definitely ascertained by experiment before use in construction, and does not vary with the length of span or with their location. Therefore, it may be the fixed quantity in the specification covering all spans.

Having established, as fixed, the strength of the material used, it is necessary that all forces to which the material may be subjected in the structure be equated to those under which the material was tested. As tests are usually made with quiescent loads, they correspond directly with the dead-load strains of the structure. The live-load strains, however, because of impact, oscillation, fatigue by repeated application, or other uncertainties, produce greater effect than the calculated strain would indicate, and must be augmented so that the increased result will represent a dead strain equivalent in its destructive effect to the live strain first calculated.

It will be recognized that the only reason why long and short spans have received different treatment heretofore has been because of the difference in the action of the live load on spans of different length, as the quality and strength of the material, and the effect of dead-load strain, are the same on whatever length of span. If, now, special attention be given to this difference of live load, provision may at once be made for this varying element. It seemed advisable, therefore, to outline a specification on the basis of dead-load strain only, and to equate all live-load strain, by changing it to that of a dead load of equivalent effect.

While the adaptation of this method of proportioning to long spans, and its use with high allowable unit strains is extreme, the principle is not new. It was in 1887 that C. C. Schneider, Past-President, Am. Soc. C. E., wrote his bridge specification for the Pencoyd Iron Company, in which he increased the live load by using an impact formula, and then allowed the same unit strains for both live and dead loads. The present suggestion is in the same direction, but proceeding one step farther. The formula for "equivalent static strain" will cover more uncertainties than the impact formula, and, therefore, should produce greater augmentation of the live load. At the same time, it will permit the use of a correspondingly greater allowable unit strain. With a proper increase of the formula, and of the allowable unit strain, it becomes applicable to long-span bridges, and thus covers the broader field of bridge construction.

It will be noticed that this method, by which the live-load strains are increased (instead of the old method, by which the allowable unit strains for live load were decreased), will produce greater counter-strains, and thus, by the better distribution of metal, will provide to some extent for the increased live load to which every important bridge may be subjected. The material in the structure will be distributed in the most economical manner possible; that is, only such as is absolutely necessary will be used to support the dead weight, while the remainder of the material is distributed so as to withstand best the live-load action—the destructive element in bridge service.

The selection of a formula for transforming the live strain into an "equivalent static" strain is the most complicated and the least satisfactory step in this method of design. It emphasizes the lack of definite information on the subject. This indefiniteness, however, is found in every method of design. It is no worse, either in character or in amount, when considered by itself, than when taken in connection with the dead load at every step in the design of spans of various lengths.

In reviewing the conditions to be covered by the formula, it is noted that a load instantaneously applied produces, with impact, double the strain of the same load at rest, and that by applying the load gradually the impact will be decreased. From this it is recognized that the impact effect of moving loads will be greater for short spans than for long ones.

Cars moving across a bridge rapidly will produce oscillation or vibration of the structure. This may vary with the speed of the moving car, the roughness of the track, the rigidity and adjustment of the structure, and possibly with the length of span.

Experiments show that a strain which is applied in continuous repetitions produces an effect about twice as destructive as a single application of the same strain. If the repetitions are not continuous, and the material is permitted to recover itself between applications, there will be a corresponding increase in the resisting power of the material. This has been described as the "fatigue of metals."

It is also noted that roadway or pedestrian traffic may become congested by panic or other cause. This congestion may be greater over small areas than over large ones—heavier per square foot on short than on long spans.

These are some of the uncertainties of live load which make it necessary to give special consideration to the resulting strains. This, as already described, may be done either by augmenting the live load, or by decreasing the allowable unit strains for the calculated live load. It is sometimes accomplished by a combination of these two methods. As the history of every important bridge shows that it is eventually loaded much in excess of the load for which it was designed, it seems preferable to augment the live-load strains rather than decrease the allowable unit strains.

The difficulty in selecting a formula arises from lack of information from which one might be produced. The experiments of J. E. Greiner, M. Am. Soc. C. E., are probably more complete than any yet made, but they do not justify any definite deductions. The effect of oscillations due to roughness of track is unknown, and, in considering loads of pedestrians and vehicles on highway bridges, the likelihood of local congestion over small areas is a matter of individual judgment, based on experiments showing widely differing results.

It seems necessary, therefore, to select some formula and then judge as to how closely its results correspond to what has heretofore been considered good practice, or to such experiments as we already have, leaving to the future such modification as further experiment or better reasoning may dictate.

It is evident that when the span is very long the application of the load is so gradual, and so rarely reaches the maximum, that any increase of live strain is unnecessary. On the other hand, for very short spans, impact alone will increase the strain 100%, and a further allowance should be made for the comparatively frequent repetition of maximum load and for other uncertainties. The impact formulas already in use may be kept in mind, but it must be recognized that the new formula for "static equivalent," which provides for more irregularities than mere impact, must give correspondingly greater increase of strain to produce the "static equivalent."

At what length of span the formula should begin to apply, is a matter of judgment; after conference,* it was assumed at 1 000 ft. It was also assumed that for very short spans the increase should not be less than 125 per cent. For intermediate spans, there seemed to be no better guide than the weights of the bridges already in service, or the results of the experiments by Mr. Greiner on the Baltimore and Ohio Railroad. On the diagram, Fig. 7, the results of these experiments have been plotted, and also various curves, for purposes of comparison. The curve, $S = 125 - \frac{1}{8}\sqrt{2\,000\ L - L^2}$, appears to give the best results, except for the span of 210 ft. On this particular span Mr. Greiner considers that the test may have been unreliable. This formula was accordingly adopted.

Allowable Strains.—If, by the formula for "static equivalent," provision is made for the most extreme loads that may occur, and if the assumed dead load is such that it will at no time be exceeded, we may conservatively use for dead load a unit strain equal to one-third of the ultimate strength, or two-thirds of the yield point, of the material. For medium steel of 60 000 lb. ultimate strength, the allowable dead-load unit strain would be 20 000 lb. per sq. in., in tension. The strains of different character, or on different material, would be deduced accordingly.

The entire specification was worked out on this dead-load unit basis, and even the foundations have been included in this method, permitting high unit pressures for only the most favorable conditions of quiescent load, and making provision for live load by use of the formula, as already mentioned.

The allowable unit strains and pressures are shown in kilo-pounds, or kips, indicated by 1 k. = 1 000 lb. The word "stress" has been avoided, because, outside of the class-room, it seems to lead to confusion. The writer has preferred the term "strain" for force, and the term "deformation" for change in dimension. The words "stress" and "strain" seem too nearly alike to have distinctive meanings, and on this account will probably continue to be used indiscriminately. For those who may prefer the word "stress," the change is easily made.

* In formulating these specifications, the writer conferred freely with a number of professional friends (C. C. Schneider, J. E. Greiner, Mansfield Merriman, J. R. Worcester, F. C. Kunz, and P. L. Wölfel, Members, Am. Soc. C. E.), and many of the best points in the specifications are credited to their suggestions.

For columns, the Rankine formula has been used, as its derivation is more rational than the right line, and for the specimen tests shown on Figs. 8 to 18, the Rankine formula is much more satisfactory. A fine line is shown on several of these diagrams to indicate the formula of the specifications.

The tests made at Watertown Arsenal in 1908 and 1909 do not include long columns, and they indicate that the pin ends were somewhat fixed, possibly by pin friction, except the two tests of 10 100 and 13 130 lb. per sq. in. ultimate strength. These two tests are close to the established formulas.

For the bearing value of piles, it is well known that any formula is, at best, a rough approximation; yet it seems necessary to formulate some rule to assist the judgment. Piles may be divided into three classes, according to the method of driving or of resistance: 1st. Where the pile supports by end bearing alone, the resistance is the bearing area only. The strength of the pile, as a column, must not be exceeded. 2d. Where the pile is driven by blows of a hammer of definite weight, falling an ascertained height, the resistance may be calculated by the force required for penetration. Experience indicates that the actual resistance, after rest, usually exceeds that which is thus calculated. 3d. Where the method of driving prevents calculation, it is necessary to estimate the skin friction between the pile and the surrounding soil. This skin friction varies with different soils. It is proportional to the area of the exposed surface and to the pressure of the soil against the pile; this pressure, in turn, varies with the depth. To this skin friction may be added the resistance, if any, of the bearing end of the pile.

Live Loads.—Having adopted a formula for transforming live strain into an "equivalent static strain," and having also adopted the unit strains which may be allowed for dead load, it now becomes necessary to ascertain the live loads to which the structure may be subjected, bearing in mind that this live load will be such as may occur over spans of more than 1 000 ft., and that for shorter spans it will be properly augmented according to the formula for "static equivalent."

For railroad trains, the distribution proposed by Theodore Cooper, M. Am. Soc. C. E., has been very generally accepted in practice, and for this specification the concentrations corresponding to what is known as "*E*-50," with a train of 5 000 lb. per lin. ft. of track, have been adopted.

For elevated railway and subway trains, the weights in service in New York City are given in Table 1.

TABLE 1.—WEIGHTS OF CARS, ETC.

	Wooden motor car.	Steel motor car.	Trailer.
Length of body	49 ft. 2 in.	49 ft. 2 in.	46 ft. 7 in.
Distance between centers of trucks	33 " 6 "	33 " 6 "	32 " 3 "
Truck wheel base	6 " 4 "	6 " 4 "	6 " 0 "
Distance between centers of wheels of adjoining cars.	9 " 4 "	9 " 4 "	9 " 4 "
Weight on front truck	52 100 lb.	54 600 lb.	29 500 lb.
Weight on rear truck	38 900 "	41 400 "	29 500 "
Total weight	91 000 "	96 000 "	59 000 "
Weight, per linear foot	1 820 "	1 920 "	1 200 "

The elevated railway cars of the Brooklyn Rapid Transit Railroad weigh about 5 000 lb. less than those in Table 1, and the cars of the Long Island Railroad and the Hudson and Manhattan Railroad weigh about 5 000 lb. more. Trains at present are made up of at least one trailer to two motor cars, but, in future, may be composed of motor cars exclusively.

Although a long-span bridge might be loaded with a continuous line of trains on all tracks, it seems hardly possible that they could all be loaded to the maximum at the same time, and the occurrence would be so rare that a load of 2 000 lb. per lin. ft. on all tracks, for spans of 1 000 ft. or more, would cover all contingencies. The local concentration of 27 300 lb. may readily be increased by a change of motor, or by an increased length of car, and should be assumed at 30 000 lb. each. All loads should be augmented by the formula for static equivalent.

For trolley cars, the weights are as follows:

Four-wheeled car, 34 ft. 0 in. long, 29 000 lb. on one truck with 4 ft. 6-in. wheel base........ 853 lb. per lin. ft.

Eight-wheeled car, 47 ft. 0 in. long, 60 000 lb., equally distributed on two trucks, 20 ft. 0 in. from center to center; wheel base of truck 4 ft. 0 in..............................1 277 " " " "

Ash car, 38 ft. 0 in. long, 100 000 lb., equally distributed on two trucks, 23 ft. 6 in. from center to center; wheel base of truck 6 ft. 6 in..............................2 632 " " " "

The weight of the ash car is unusual, and should be provided for only incidentally. The weight of the eight-wheeled passenger car is rather less than is sometimes found. It would seem advisable, in order to provide for some increase, to adopt for long-span bridges a uniform load of 1 500 lb. per lin. ft. of track, and also make provision for a single ash car—the loads for short spans to be properly augmented by formula.

The loads on the roadway of a bridge will differ materially from those which are likely to occur above a subway. A driver with an unusually heavy load will not cross a bridge without careful inquiry as to its strength. On the other hand, any load which is hauled through the streets may, unknowingly, be hauled over a subway. For the former, therefore, a reasonable load may be permitted, and the bridge may be protected against excessive weights, while, for the latter, all loads which may occur must be considered.

The truck loads which have occurred in New York City have been ascertained as follows:

1. Cables weighing 84 tons are carried equally distributed on four wheels, the axles 12 ft. 0 in. from center to center, with a wheel gauge of 8 ft. 0 in. The weight of the truck will add 6 tons, and 16 horses, at 1 500 lb. each, will weigh 12 tons more. The length of the truck is 16 ft. and that of eight teams of horses will be 96 ft., making a total of 112 ft.

2. Girders weighing 65 tons are carried mainly on two rear wheels of a long truck. The weight of the truck will add 2 tons for each pair of wheels; the gauge is 8 ft. 0 in. The length of the truck is 40 ft. and that of five teams of horses is 60 ft., making a total of 100 ft.

3. An automobile truck will carry 10 tons on four wheels. The weight of the truck will be 6 tons additional. The axles are 16 ft. apart, and the length of the body is 20 ft. This load averages 1 600 lb. per lin. ft.

4. The R. H. Howe Company has carried 75 tons on a truck weighing 10 tons, most of the load being on the two rear wheels. This load is hauled by 16 horses.

5. A coal truck weighing 2½ tons will carry a load of 7 tons, and will be hauled by three horses weighing 2½ tons more. The length of the truck is 12 ft., the gauge being 8 ft. This load averages 1 000 lb. per lin. ft.

6. An asphalt truck has about the same dimensions and weight as a coal truck.

7. The trucks which carry the Lidgerwood hoists, take a load of 15 tons. The weight of the truck is 3 tons, and that of four horses is 3 tons more. The length of the truck is 15 ft., and that of two teams of horses may be taken at 24 ft.

8. The standard truck for general use carries a load of 5 tons, weighs 2½ tons, and is hauled by two horses weighing 1½ tons more. The length of the truck and pole is 24 ft., and the weight averages 750 lb. per lin. ft.

These loads are summarized in Table 2.

TABLE 2.—WEIGHTS OF TRUCKS.

Kind of truck	Total load, in tons.	Load, in pounds per linear foot.	Load, in pounds per square foot.
Cable truck	102	1 800	150
Automobile truck	16	1 600	135
Girders on truck	76	1 500	125
Lidgerwood truck	21	1 150	95
Coal truck	12	1 000	85
Asphalt truck	12	1 000	85
Standard truck	9	750	65

Those of the loads in Table 2 which exceed 85 lb. per sq. ft. are for special trucks, and rarely occur. On a long span, probably half the trucks would be empty. The majority would be "standard" or light delivery wagons; yet, in the case of a large conflagration in the city, the span might be packed with people. It would thus seem that 80 lb. per sq. ft. would be a maximum load, if indeed it should not be much less, for long spans.

The Board of Engineers for Manhattan Bridge recommended, for local concentrations, 24 tons on two axles spaced 12 ft. apart, with a 5-ft. gauge, and assumed to occupy a width of 12 ft. and a length of 30 ft. A concentration of 15 tons on one axle, with an 8-ft. gauge, may be taken as an equivalent, and more general in its application, as loads are rarely distributed equally on four wheels, but, as shown in Table 2, are more often carried on one axle. The local concentrations of cable trucks, and of heavy girder trucks, are so excessive and so unusual that the loads should be floated on barges to a point sufficiently near the place of delivery to avoid crossing bridges.

Street and sidewalk surfaces, in addition to the loads above mentioned, may support a pile of paving blocks or heavy merchandise. This is usually estimated at from 500 to 1 000 lb. per sq. ft. on the roadway; but, as heavy concentrations are already specified for the roadway, and the quiescent load on the footwalk, is, for uniformity of specification, augmented by formula, a provision of 300 lb. per sq. ft. would seem proper. For heavy concentrations, provision is made for distribution on subways by the soil of varying depth.

Centrifugal Force.—The speed of railroad trains on curves on bridges is usually estimated at 40 miles per hour, or 60 ft. per sec. The probability of this speed obtaining on all tracks at the same time, is very remote.

The formula for centrifugal force is:

$$C = \frac{V^2}{32.2\,R}\,W$$

Where,

C = the centrifugal force;
V = the velocity, in feet per second;
R = the radius of the curve, in feet;
and W = the weight of the train.

For 1° of curvature this formula becomes:

For 40 miles per hour	(60 ft. per sec.)	...	$C = 0.0195\ W$
" 30 " " "	45 " " "	...	$C = 0.011\ W$
" 20 " " "	30 " " "	...	$C = 0.0049\ W$
" 10 " " "	15 " " "	...	$C = 0.0012\ W$

For curves of less radius than 5 729 ft. (1°) the centrifugal force will vary inversely as the radius, or approximately in direct ratio with the degree of curvature.

Wind Pressure.—The velocity and pressure of the wind on bridges has been a subject of considerable investigation, but of very unsatisfactory determination. Records of severe storms have shown pressures varying from 40 to 80 lb. or more per sq. ft., but it is generally conceded that these great pressures are of short duration and of small area. For a long time, the Tay Bridge was a nightmare to bridge engineers, but it seems to have been well established that it was destroyed, not by excessive wind pressure, but by accident, or possibly by defect in construction. The wind pressure at the time appears to have been considerably less than 30 lb. per sq. ft. The late C. Shaler Smith, M. Am. Soc. C. E., found only one case where a pressure of 30 lb. per sq. ft. exceeded 60 ft. in width, and, in the case of a 320-ft. span, designed by him for 30 lb. per sq. ft., with 20 000 lb. per sq. in. allowable strain on wrought iron, a tornado showing a pressure of 52 lb. per sq. ft. on the bridge, and 84 lb. pressure elsewhere, did not destroy the span. At Stamford, Conn., in 1904, the writer witnessed a tornado which uprooted trees and swept a clean path, about 100 ft. wide, across the city, but was hardly felt elsewhere. The provision of 30 lb. per sq. ft., therefore, would seem to make ample provision for safety.

Sliding Friction.—The coefficient of sliding friction for tractive power may be assumed as 20% or more, but emergency-brake tests on especially equipped trains show from 12 to 16%, and even this would probably never occur on all trains at the same time on a long-span bridge fully loaded. Considering the improbability of its occurrence, and the fact that the allowable strain is well within the yield point, a coefficient of 10% would seem to make ample provision.

Reinforced Concrete.—The specifications covering the design of reinforced concrete are a concise abstract of what may be considered general practice. Preference has been given to deformed bars as a factor of safety.

The writer is indebted to A. J. Malukoff, Assoc. M. Am. Soc. C. E., Assistant Engineer, Department of Bridges, New York City, for his thorough work in compiling tests and other information, and to Mr. A. I. Raisman, Assistant Engineer, Public Service Commission, for his deduction of the formula for the permissible pressure on piles.

With this general, though brief, review of the specifications, they are presented to the Society for such use or discussion as may be desired. They have been in regular service about 3 years, and little modification has been suggested.

SPECIFICATIONS FOR THE DESIGN OF BRIDGES AND SUBWAYS.

LOADS.

The structure shall be designed to resist the strains* due to the following loads:

Dead Load.—The dead load shall consist of the entire weight of the structure. In the case of subways, it shall include the weight of the superimposed earth and, when under private property, such loads as are provided for by the Building Law.

The strains computed therefrom will be considered as static.

The weight of the materials shall be estimated as follows:

Material.	Weight, per cubic foot.
Steel, rolled and cast	490 lb.
Iron, wrought	480 "
Iron, cast	450 "
Earth	100 "
Timber, untreated	48 "
Timber, treated	60 "
Masonry (average)	150 "
Buildings, per floor. See Building Law.	

Live Load.—The live load shall consist of the moving load plus the increase by formula for equivalent static strain given herein. The resulting strains shall be considered in effect as equivalent to a static strain of like amount.

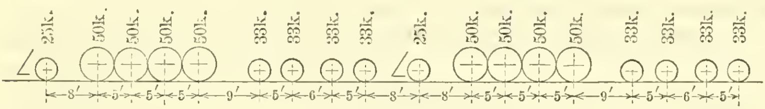


FIG. 1.

Moving Load (1 k. = 1 000 lb.).—The moving loads shall be assumed as follows:

Railroad trains on bridges shall be estimated as a continuous train of 5 000 lb. (5 k.) per lin. ft. of each track headed by two typical engines with wheel loads as shown by Fig. 1.

Elevated or subway trains shall be estimated as continuous at 2 000 lb. (2 k.) per lin. ft. of each track, or as a local concentration of two adjacent motor trucks with axle loads spaced as shown by Fig. 2.

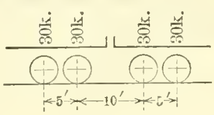


FIG. 2.

* The term "strain" is here used for a force, and the term "deformation" for change of dimension.

Trolley cars shall be estimated as continuous at 1 500 lb. ($1\frac{1}{2}$ k.) per lin. ft. of each track, or as a local concentration of one ash car with axle loads spaced as shown by Fig. 3.

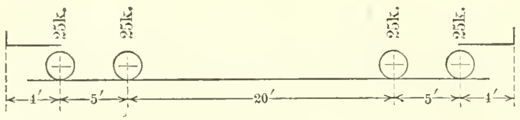


FIG. 3.

On subways, provision shall be made for a continuous line of ash cars.

The roadbed for trolley cars on bridges shall be assumed as 12 ft. wide, and shall be capable of carrying the loads specified for the roadways of bridges.

The roadway load for bridges shall consist of a uniform load of 80 lb. per sq. ft. of surface, or a local concentration of 30 k. on one axle, with a wheel gauge of 8 ft. This load may be assumed to cover a space 12 ft. wide and 40 ft. long. On subways it shall consist of a uniform load of 300 lb. per sq. ft. of surface, or a local concentration of 100 k. on four wheels, 12 ft. between axles, and 8 ft. gauge.

Footwalks for bridges and in subways shall be estimated as loaded at 80 lb. per sq. ft. of surface. Sidewalks over subways shall be estimated at 300 lb. per sq. ft. of surface.

When concentrated loads are carried on subways they may be assumed to be distributed over an area 2 ft. square on the pavement and thence through the earth at a slope of $\frac{1}{2}:1$.

The centrifugal force shall be estimated by the following formula:

$$C = 0.020\ W\ D;^{*}$$

Where,

C = Centrifugal force,
W = Weight of moving cars,
D = Degree of curvature.

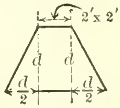


FIG. 4.

All the moving loads above specified shall be increased according to their various lengths by the following formula to produce the static equivalent:

$$S = 125 - \frac{1}{8}\sqrt{2\,000\,L - L^2}$$

($S = 0$, when $L > 1\,000$ ft.)

* Reduce this coefficient by 0.001 per degree of curvature from $D = 3°$ to $D = 20°$. For sharper curves use the coefficient for a 20° curve.

Where,

S = Increase, in percentage;
L = Length, in feet, of applied loading which produces maximum strain in the member. Not to exceed 1 000 ft.

Provision shall be made for a wind pressure, acting in either direction, horizontally, of 30 lb. per sq. ft., of the surface as seen in elevation, of the floor system, considering ties, etc., as solid area, of one truss complete, and of one-half the surface of all remaining trusses, as well as that of a train 14 ft. high and 500 ft. (or less) in length. The pressure on the train, as well as one-half the pressure on the structure shall be considered as moving load.

Provision shall be made for the sudden starting or stopping of a train 500 ft. in length, estimating the coefficient of sliding friction at 10 per cent.

There will be no increase by formula for static equivalent to either the longitudinal or wind forces.

On bridges, provision shall be made for a variation in temperature of 120° Fahr. (A difference of 40° in the temperature of the chords of the same truss, or in that of adjacent trusses of the same structure shall be considered in spans of more than 300 ft.)

In calculating the shearing strains on bridges, from uniform moving loads, the panel lengths shall be considered as fully covered, but the half-panel load at the head shall be neglected.

Allowable Static Strains.

The static strains from dead load and the equivalent static strains from moving load shall be combined, and the maximum strains in either direction deduced therefrom.

Members subject to alternate strains of tension and compression shall be proportioned for the strain giving the largest section, and designed to resist each strain. If the alternate strains occur in immediate succession (as when produced by moving load), each strain shall be increased by 50% of the smaller. The connections, in either case, shall be proportioned for the arithmetical sum of the strains.

The allowable static strain per square inch for the total combined loading shall in no case exceed those given in the following tables.

In all cases, however, where the structure takes any building load, the design shall be made in accordance with the Building Law.

When beams and girders are embedded in concrete, the allowable flange strain may be increased 25 per cent.

In case of field rivets, 25% excess will be added to the number of rivets required as above. (When machine-driven, this may be reduced to 15% excess.)

Allowable Static Strains for Steel, Iron, and Reinforced Concrete.

In units of 1 000 lb. = 1 k. per sq. in.

All live strains must first be increased to equivalent static strains by formula.

Nature of strain.	Steel wire.	Nickel.	Steel.		Iron.		Concrete, reinforced.
			Medium structural.	Cast.	Wrought.	Cast.	
Tension. (Net)	75 k.	30 k.	20 k.	16 k.	16 k.	4 k.	0.07 k.
Compression. (1 diam.) (Gross)	....	30 k.	20 k.	20 k.	16 k.	20 k.	....
Compression. (8 diams.) "	....	....	....	....	....	....	0.50 k.
Compression. (12 diams.) "	....	24.7 k.	16.5 k.	....	13.2 k.	....	....
Compression. Columns*	....	$\dfrac{30\text{ k.}^*}{1+\dfrac{l^2}{8\,000\,r^2}}$	$\dfrac{20\text{ k.}^*}{1+\dfrac{l^2}{8\,000\,r^2}}$	....	$\dfrac{16\text{ k.}^*}{1+\dfrac{l^2}{8\,000\,r^2}}$	$\dfrac{20\text{ k.}^*}{1+\dfrac{l^2}{1\,000\,r^2}}$	$\dfrac{0.550\text{ k.}^*}{1+\dfrac{l^2}{600\,l^2}}$
Bending. (Beams, outer fiber)	....	30 k.	20 k.	....	16 k.	....	0.60 k.
Bending. (Pins, rivets, and bolts)	....	45 k.	30 k.	....	24 k.	....	....
Shear. (Pins, rivets, web) (net section)	....	22 k.	15 k.	16 k.	12 k.	4 k.	0.07 k.
Bearing. (Pins, rivets, and bolts)	....	45 k.	30 k.	....	24 k.	....	....
Bearing. (Roller.) Per linear inch	....	1.0 k. d	0.75 k. d	....	0.5 k. d	....	....
Adhesion. Concrete to steel. (Deformed bars)	....	....	....	....	....	....	0.10 k.

* Compression members in steel and iron shall not receive greater unit strain than that allowed for 12 diameters, and in reinforced concrete not greater than that allowed for 8 diameters.

l = Length of column, in inches;

r = Least radius of gyration of cross-section, in inches;

d = Diameter of roller, in inches;

l = Least thickness, in inches.

When tension on rivets is unavoidable, the tensile component shall be considered as having double the effect of an equal amount of true shear.

In designing the invert, the live load on top of the subway may be neglected where the loads on columns and walls are distributed over the whole invert.

Allowable Static Strains for Timber.

In units of 1 000 lb. = 1 k. per sq. in.

All live strains for timber must first be increased to equivalent static strains by one-half the amount indicated in the formula already specified.

Nature of strain.	White oak.	White pine.	Georgia pine.	Spruce.	Hemlock.
Tension, with grain	1.5 k.	1.0 k.	1.8 k.	1.2 k.	0.9 k.
Tension, across grain	0.3 k.	0.075 k.	0.09 k.	0.075 k.	
Compression, end bearing	2.1 k.	1.6 k.	2.4 k.	1.8 k.	
Compression, to 15 diameters	1.3 k.	1.0 k.	1.5 k.	1.2 k.	1.2 k.
Compression, columns*	$\dfrac{1.6\text{ k.}^*}{1+\dfrac{l^2}{1\,000\,d^2}}$	$\dfrac{1.2\text{ k.}^*}{1+\dfrac{l^2}{1\,000\,d^2}}$	$\dfrac{1.8\text{ k.}^*}{1+\dfrac{l^2}{1\,000\,d^2}}$	$\dfrac{1.4\text{ k.}^*}{1+\dfrac{l^2}{1\,000\,d^2}}$	$\dfrac{1.4\text{ k.}^*}{1+\dfrac{l^2}{1\,000\,d^2}}$
Compression, across grain	0.75 k.	0.3 k.	0.5 k.	0.3 k.	0.2 k.
Shear, across grain	1.5 k.	0.75 k.	1.9 k.	1.1 k.	0.9 k.
Shear, with grain	0.3 k.	0.15 k.	0.2 k.	0.15 k.	0.15 k.
Bending, outer fiber	1.5 k.	1.0 k.	1.8 k.	1.0 k.	0.9 k.
Modulus of elasticity	1 100 k.	1 000 k.	1 700 k.	1 200 k.	900 k.

* Compression members of timber shall not receive greater unit strain than that allowed for 15 diameters. When designed to take shear with the grain, selected timber shall be used.
l = Length of column, in inches;
d = Least diameter of column, in inches.
In this table the values for allowable static strains for timber are 50% greater than those formerly used for miscellaneous loading without impact.

Allowable Static Pressures.

The following allowable pressures are for static loads only. All live loads shall first be increased to equivalent static loads by formula.

Allowable Static Pressures on Masonry.

Granite (cut stone)	80 k. per sq. ft.
Limestone (cut stone)	60 k. " " "
Sandstone (cut stone)	40 k. " " "
Rubble	30 k. " " "
Concrete (1:3:6)	50 k. " " "
Concrete (1:2:4)	60 k. " " "
Brickwork, Portland cement	40 k. " " "

For a wall of height, h, exceeding eight times its thickness, t, the above pressures, P, should be reduced by the following formula:

$$p = \frac{P\,(1.1)}{1 + \dfrac{h^2}{600\,t^2}}.$$

Allowable Static Pressures on Soils.

Sound ledge rock....................	150 k. per sq. ft.
Hardpan, or compact gravel..........	20 k. " " "
Coarse sand, or gravel................	12 k. " " "
Clean sand, or dry clay..............	8 k. " " "
Clay, moist..........................	4 k. " " "
Silt	2 k. " " "

The above allowable static pressures are for the most favorable conditions. For questionable soils these pressures should be reduced 50 per cent. In deep foundations, friction and buoyancy may be allowed for in computation.

Piles—Designed for 40 k. Each (Static).—The actual sustaining value, in equivalent static load, shall be ascertained in the field as follows:

1st.—When a pile is driven to refusal, and supports by end bearing, without side friction, allow 1 400 lb. (1.4 k.) per sq. in. of net section, properly reduced for column length when not stayed.

2d.—When driven to resistance by hammer blows: $p = \frac{3\,W h}{d + 1}$.

3d.—When driven by water-jet, or by hammer, in soft material:

$$p = (S\ a + R)\ l.$$

p = Safe static load per pile, in pounds;
W = Weight of hammer, in pounds;
h = Height of fall of last blow, in feet;
d = Penetration under last blow, in inches;
S = Skin surface of pile under soil, in square feet;
l = Length of pile in ground, in feet.

$S\,a\,l$, represents the resistance due to skin friction.

$R\,l$, represents the value of end bearing for a diameter of 12 in. at the point of the pile.

For loam and silt.............	a = 1.7......	R = 47
" moist clay................	a = 7.4......	R = 96
" sand, or clay.............	a = 19.2......	R = 203
" coarse sand and gravel....	a = 29.0......	R = 300

Details of Design,

For Structural Steel and Iron.

Spans for Calculation.—The assumed spans for calculation shall be as follows:

Pin-connected trusses..	distance between centers of end pins.
Riveted girders.......	distance between centers of bearings.
Cross-girders	distance between centers of trusses or columns.
Stringers	distance between centers of cross-girders.
Cross-ties and flooring.	distance between centers of stringers.

Depth for Calculation.—The assumed depth for calculation shall be:

Pin-connected trusses..distance between centers of chord pins.
Riveted-lattice trusses.distance between centers of gravity of chords.
Plate-girders ....distance between centers of gravity of flanges.
(Not to exceed out to out of flange angles.)

Lateral Bracing.—Each main panel of deck bridges shall be provided with intermediate sway-bracing of a sufficient section to carry one-half the maximum increment due to wind on train, and to centrifugal force. The end sway-bracing shall be proportioned to carry the lateral strain to the support.

Through bridges shall be provided with post-brackets, at the intermediate panel points, of sufficient strength to maintain the panel in a vertical position under the specified wind pressure; or, when the height of the top chord exceeds 20 ft. above the floor, an overhead system of sway-bracing shall be used.

Rigid lateral bracing will be preferred.

The value of $\frac{l}{r}$ for rigid members shall not exceed 200.

Where adjustable rods are used, check-nuts will be provided, and the rod shall be designed to receive an initial strain of 10 k.

Design of Details.—The use of rolled material which exceeds $\frac{5}{8}$ in. in thickness, and is to be punched, shall be generally avoided.

All parts shall preferably be designed so that the strains coming on them may be calculated definitely.

The center line of resistance of a member will be along its neutral axis, and connections shall be designed so as to avoid bending the member or its details.

The line of strain shall pass centrally through any cluster of rivets which resist it, and where angles or plates are connected in any other manner, proper provision shall be made for the moments and secondary strains produced.

Details shall be designed so as to give free access for inspection and painting, and water-pockets shall be avoided.

In no case shall the details be of less strength than the member as designed.

All members which are subject to direct strains combined with bending shall be proportioned so that the greatest fiber strain will not exceed the specified allowable strain, properly reduced in the case of columns.

In continuous upper chords of deck bridges carrying the floor, the bending strain due to the live load shall be computed from a moment equal to three-quarters of the maximum moment produced on a span of one panel considered as a simple beam.

The strain on the outer fiber of solid shapes shall be computed from the moment of inertia of the section.

Plate Girders.—When the web-plate is not spliced, one-eighth of its gross section may be considered as flange area.

No allowance shall be made for the web, when spliced, in calculating the flange section of plate girders.

At least one upper flange plate, when used (except when embedded in concrete), shall extend from end to end of the girder. Any additional plates used to make up the flange section shall be of such lengths as to allow at least two rows of rivets of the regular pitch being placed at each end of the plate beyond the theoretical point required, and there shall be a sufficient number of rivets at the ends of the plate to transmit their value before the theoretical point of the next outside plate is reached.

Where the flange plates vary in thickness, they shall decrease outward from the flange angles.

The total thickness of plates and angles shall not exceed five times the diameter of the rivets used.

All flange plates, subject to either tension or compression, spliced in the length of the girder, shall be properly covered with an extra amount of material equal in section to the material spliced, with sufficient rivets on either side to transmit the strains from the parts cut.

The upper flanges shall be braced at intervals not exceeding sixteen times the width. Where this is impracticable, the allowable compressive flange strain shall be reduced in accordance with the following formula:

$$p = \frac{P}{1 + \frac{L^2}{1\,200\,W^2}}$$

Where,

p = The reduced strain per square inch allowable;

L = The unbraced length of flange, in inches;

W = The width of flange, in inches.

Values of P = 20 k. for medium steel and 16 k. for wrought iron.

In calculating the shearing or bearing strain in web rivets of plate girders, the whole of the shear acting on the side of the panel nearest the abutment shall be considered as being transferred into the flange angles within a distance equal to the depth of the girder.

The webs of plate girders, wherever cut, shall be spliced by a plate on each side of the web capable of transmitting the full shearing strain through splice rivets.

When the thickness of the web is less than one-fiftieth of the unsupported distance between flange angles, stiffeners shall be riveted on both sides of the web, with the outstanding leg as long as the flange angles will allow. The distance from center to center of stiffeners, generally, shall not exceed the depth of the full web-plate, but they shall in no case be closer than 4 ft., nor farther apart than 6 ft. Web-plates, generally, shall have stiffeners at all splices, at points of concentrated loading, and at each end of bearing plates.

Net Sections.—Net sections shall be used in all cases in calculating tension members, and, in deducting rivet holes, they shall be taken as $\frac{1}{8}$ in. wider than the nominal diameter of the rivet.

In calculating the net sections having rivets staggered, all rows shall be deducted unless arranged so that the net section along a zigzag line, taking all distances in a diagonal direction at only three-fourths of their value, exceeds the corresponding net section across the plate.

Pin-connected, riveted, tension members shall have a net section back of the pin-hole, parallel to the axis of the member, of not less than the required net section of the body of the member, and shall have a section through the pin-hole at least 25% in excess of that required section.

Rivet Spacing.—Rivets shall not be spaced closer than three diameters from center to center, nor farther apart in the direction of the strain than sixteen times the thickness of the thinnest external plate connected, and not more than thirty-two times that thickness, at right angles to the line of strain.

Rivets shall not be spaced closer to the sides of plates than $1\frac{1}{4}$ diameters to the center of the rivet, nor farther from the side than eight times the thickness of the plate. In no case shall the pitch of rivets exceed 6 in.

Field rivets shall be reduced to a minimum.

Built chords, when faced for bearings, shall be spliced on four sides sufficiently to hold the abutting members accurately in place and to transmit 50% of the strain through the splice-plates. All other joints shall be fully spliced.

Where strain is to be transmitted through idle material, the latter shall be extended for direct riveting.

When necessary to obtain sufficient bearing surface at pin-holes, reinforcing plates shall be added. These plates shall be designed to distribute their proportion of the bearing strain from the pins to the member to which they are connected.

Latticing Compression Members.—All segments of members in compression, connected by latticing only, shall have batten-plates at each end, the thickness of which shall be not less than one-fortieth of the distance between the rivets connecting them to the compression member. In no case shall the length of the batten-plates be less than $1\frac{1}{4}$ times the width of the member.

The pitch of rivets at the ends of built compression members shall not exceed 4 diameters for a distance $1\frac{1}{4}$ times the width of the member.

The distance between the connections of latticing shall be such that the individual members between them, composing the column, shall be stronger than the column as a whole, and in no case shall this distance exceed 8 times the least width of these members. Where the ends of the compression member are forked to connect with pins, the combined strength of these legs shall be, at least, twice the strength of the column, and the reinforcing plates shall extend not less than 6 in. beyond the edge of the batten-plates. As much of the column flange as is practicable shall extend the full length of the reinforcing plates.

Single lattice bars shall generally be inclined at an angle of 60° to the axis of the member, and double lattice bars at an angle of 45°, with a rivet at their intersection.

Single lattice bars shall have a thickness of not less than one-fortieth, and double lattice bars not less than one-fiftieth, of the distance between the rivets connecting them to the compression member; and their width shall be:

For 33-in.	to 48-in. web,	6-in. flange	angles,	four ⅞-in.	rivets,	use 6 by 3½ by ½-in. angles.
" 20 "	" 32 "	" 4 " "	"	two ⅞ "	"	" 5½-in. strap.
" 15 "	" 19 "	" 4 " "	"	one ⅞ "	rivet.	" 3½ " "
" 12 "	channels or	3 " "	"	one ¾ "	"	" 3 " "
" 9 "	" "	2½-in. "	"	one ¾ "	"	" 2½ " "
" 8 "	" "	2 " "	"	one ⅝ "	"	" 2 " "

Expansion Rollers.—All bridges more than 80 ft. long, which bear on masonry, shall be provided with pin-bearing bolsters, and at one end with turned friction rollers not less than 4 in. in diameter between two planed surfaces.

For spans of 80 ft. or less, planed surfaces shall be used without rollers.

The nest of rollers shall be designed so as to prevent displacement, and be kept dust free.

Bearings Anchored.—Trusses shall be secured against side motion on bearing plates and rollers.

The bolster blocks shall be joined to the truss, and the bearing plates shall be secured to the underlying supports by bolts or dowels.

Eye-Bar Packing.—Eye-bars shall be packed so as to produce the least bending moment on the pin, and shall not be packed out of line with the axis of the member more than $\frac{1}{8}$ in. to 1 ft.

Members packed on pins shall be held against lateral movement by filling rings. The pin shall be of sufficient length to give full bearing to all members and with screw ends extending two threads beyond the nut, to permit burring.

Minimum Section.—No exposed material less than $\frac{1}{4}$ in. thick shall be used, except for packing or other idle purposes. Counter-rods shall have not less than $1\frac{1}{2}$ sq. in. in sectional area.

Camber.—Truss bridges shall be given a camber by making the compression chord longer than the tension chord, in the proportion of $\frac{1}{8}$ in. to 10 ft.

DETAILS OF DESIGN FOR REINFORCED CONCRETE.

General Rules for Design.

Spans for Calculation.—The length of span, for calculation, shall be the distance, in the clear, plus the width required for the theoretical bearing.

Design of Details.—The covering for the reinforcing metal in beams shall be not less than 1 in. of concrete, and in floor slabs and other thin structures it shall be not less than ½ in. In all other cases it shall be not less than 1½ in., and in no case shall it be less than 1¼ times the diameter of the bar used.

Reinforcing bars shall be spaced not farther apart than 6 times the thickness of the bar (except in T-beam construction), and arranged so as to permit spading readily. Deformed bars shall generally be used, but, where plain bars are permitted, the allowable adhesive strain shall be reduced 30%, and the bars shall be spaced not closer than 4 diameters.

The location and shape of the joints at which reinforced concrete can be stopped shall be determined by the designing engineer and indicated clearly on the plans. The concrete, for any one of the portions into which the construction is thus divided, shall be deposited in one continuous operation.

Shear.—The concrete between the centers of flange strain shall be sufficient to take up the maximum shear; otherwise, straps, properly spaced, shall be supplied to make up the deficiency.

Straps, when needed, shall generally have a spacing equal to one-half to once the depth between the centers of strains, and shall be designed to take up in tension that part of the increment of the longitudinal strain which is not taken up by the concrete.

The straps shall be wrapped around the longitudinal reinforcement, and shall be designed to transfer their strains to the flanges.

The ends of alternate longitudinal reinforcement bars shall preferably be bent to make an angle of 30° with the flange, and extend to the compression flange above the support.

The strains shall be calculated in accordance with the following rules and formulas:

Proportion for mixing concrete................1 : 2 : 4
Maximum tension in steel...................$F_s = 20$ k.
Maximum compression in concrete, of beams....$F_c = 0.6$ k. per sq. in.
Maximum compression in concrete, of columns..$F_c = 0.5$ k. " " "

except that when the height of the column exceeds 8 times the least dimension, this value is to be reduced by the formula:

$$p = \frac{0.550 \text{ k.}}{1 + \dfrac{L^2}{600\, t^2}}$$

Where,

p = Maximum compression per square inch;
L = Length, in inches;
and t = Least thickness, in inches.

Steel bars in compression members, up to 2% of the cross-section of the member, may be considered to take a strain per unit equal to 15 times that permitted in the concrete. Such reinforcing bars must be properly laced at distances not greater than the least dimension of the member.

Maximum shear in concrete............	0.07 k. per sq. in.
Maximum adhesion of concrete to steel (deformed bars).....................	0.10 k. " " "
Modulus of elasticity of steel...........	30 000 000
Modulus of elasticity of concrete........	2 000 000

Bending.—To design for bending, the following assumptions will be made:

1st.—The bending strain in any fiber varies directly as the distance from the neutral axis.

2d.—The ratio between the moduli of elasticity of steel and of concrete is constant for the working strains assumed:

$$\frac{E_s}{E_c} = 15.$$

3d.—The longitudinal tensile strain of concrete, generally, shall not be considered.

4th.—Partial continuity may be assumed in the design of beams and slabs, and the moment, $M = \frac{W\,L}{10}$, may be used in place of $\frac{W\,L}{8}$, where the conditions warrant it. In any case, however, steel reinforcement should be provided over the supports to prevent cracking.

Formulas for Calculations.—

Let M = the maximum bending moment in the beam.

$$p' = \frac{A_s}{b\,d} = \frac{\text{Area of steel}}{\text{Effective area of beam}}.$$

$$e = \frac{E_s}{E_c}.$$

A_c = Area of concrete.

Bending.

Location of Neutral Axis.—The neutral axis passes through the center of gravity of the effective section, if the concrete below the neutral axis is disregarded and the area of the steel is assumed equal to e times its area of concrete; accordingly:

$$b\,x\,d\,\frac{x\,d}{2} = e\,A_s\,(d - x\,d);$$

$$\text{but } A_s = p'\,b\,d\,;$$

$$\text{therefore, } b\,d^2\,\frac{x^2}{2} = e\,p'\,b\,d^2\,(1 - x);$$

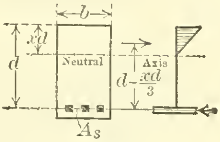


FIG. 5.

dividing both sides by $b\,d^2$, we obtain:

$$\frac{x^2}{2} = e\,p'\,(1 - x);$$

$$\text{whence } x = \pm\sqrt{2\,e\,p'\,\left(1 + \frac{e\,p'}{2}\right)} - e\,p'.$$

Note here that x depends only on e and p', and is independent of b and d. It can readily be tabulated. (See table on page 338.)

Resisting Moment of Rectangular Beams.—Having the location of the neutral axis, the resisting moment follows directly:

$$M = \frac{1}{2} f_c\,b\,x\,d\,\left(d - \frac{x\,d}{3}\right) = \frac{1}{2} f_c\,x\left(1 - \frac{x}{3}\right)\,b\,d^2 = k_c\,b\,d^2, \text{ and}$$

$$M = f_s\,A_s\left(d - \frac{x\,d}{3}\right) = f_s\left(1 - \frac{x}{3}\right)p'\,b\,d^2 = k_s\,b\,d^2,$$

$$\text{where } k_c = \frac{1}{2}\,f_c\,x\left(1 - \frac{x}{3}\right), \text{ and } k_s = p'\,f_s\left(1 - \frac{x}{3}\right).$$

The values of k_c and k_s depend only on the proportion of steel for any given value of e. Tables are given herewith for these values. Use the smaller k for the design of beams. From 0.70 to 1.50% of steel gives the usual economical sections.

Resisting Moment of T*-Beams.*—As the concrete below the neutral axis is not considered for the resisting moment of a beam, the formula for resisting moment of a T-section is the same as for an ordinary beam, providing the neutral axis passes through the flange.

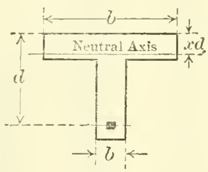


FIG. 6.

Where the neutral axis passes through the stem, a general formula is too cumbersome for use. The location of the neutral axis may be calculated as indicated above.

Shear.

$$\text{Unit shear} = \frac{\text{max. shear}}{b\,d\left(1 - \frac{x}{3}\right)} \text{ for plain reinforced beams.}$$

$$\text{“} \quad \text{“} = \frac{\text{max. shear}}{b'd\left(1 - \frac{x}{3}\right)} \text{ for T-beams.}$$

Adhesion.—

$$\text{Unit adhesion} = \frac{\text{maximum shear}}{d\left(1 - \dfrac{x}{3}\right)\text{perimeter of rod sections}}.$$

Columns.—

Maximum column load $= p\,(A_c + e\,A_s)$.

$$\text{Where, } p = \frac{550}{1 + \dfrac{l^2}{600\ t^2}}$$

l being the length, and

t the least dimension, of the column, in inches.

The maximum value of $p = 500$ lb.

TABLE OF VALUES OF x, k_c, k_s, ETC.

For $f_s = 20\ 000$ lb., $f_c = 600$ lb., $\dfrac{E_s}{E_c} = 15$, Max. shear $= 70$ lb.,

Max. adhesion $= 100$ lb.

p'	x	k_c	k_s	Maximum load per square foot on account of shear, in pounds.	MINIMUM SPAN FOR RODS.			
					¾-in.	1-in.	1¼-in.	1½-in.
0.002	0.218	61	37	8 200	10 ft.	13 ft.	16 ft.	20 ft.
0.003	0.258	71	55	5 370	10 “	13 “	16 “	20 “
0.004	0.292	79	72	4 000	10 “	13 “	16 “	20 “
0.005	0.320	86	90	3 270	12 “	16 “	20 “	24 “
0.006	0.344	91	106	3 040	10 “	14 “	18 “	21 “
0.007	0.365	96	122	2 840	10 “	13 “	16 “	20 “
0.008	0.384	101	140	2 700	9 “	12 “	15 “	18 “
0.009	0.402	104	156	2 550	8 “	11 “	14 “	17 “
0.010	0.418	108	172	2 420	8 “	10 “	13 “	16 “
0.011	0.433	112	189	2 310	7 “	10 “	12 “	15 “
0.012	0.446	114	204	2 240	7 “	9 “	12 “	14 “
0.013	0.460	116	220	2 180	7 “	9 “	11 “	13 “
0.014	0.471	119	236	2 090	6 “	8 “	10 “	13 “
0.015	0.484	121	252	2 060	6 “	8 “	10 “	12 “
0.016	0.491	123	267	2 000	6 “	8 “	10 “	12 “
0.017	0.503	126	284	1 940	6 “	7 “	9 “	11 “
0.018	0.513	127	299	1 890	5 “	7 “	9 “	11 “
0.019	0.523	129	314	1 860	5 “	7 “	9 “	10 “
0.020	0.530	131	330	1 820	5 “	7 “	8 “	10 “
0.021	0.539	133	345	1 790	5 “	7 “	8 “	10 “
0.022	0.542	134	360	1 760	5 “	6 “	8 “	9 “
0.023	0.555	135	375	1 730	5 “	6 “	8 “	9 “
0.024	0.562	137	390	1 700	4 “	6 “	7 “	9 “
0.025	0.569	138	405	1 680	4 “	6 “	7 “	9 “
0.026	0.575	139	420	1 660	4 “	6 “	7 “	8 “
0.027	0.580	140	436	1 640	4 “	5 “	7 “	8 “
0.028	0.588	142	450	1 620	4 “	5 “	7 “	8 “
0.029	0.595	143	465	1 590	4 “	5 “	6 “	8 “
0.030	0.600	144	477	1 570	4 “	5 “	6 “	8 “

“Minimum Span for Rods” gives the smallest spans where given rods may be used for uniform load on account of adhesion to rods.

“Maximum load per square foot” gives the maximum load that may be used for uniformly-loaded beams, on account of shear of concrete.

Allowable Compressive Strains For Steel, Wrought-Iron And Cast-Iron Columns

The quantities below give the proportion of allowable compressive strain per square inch of cross-section for the different values of l/d, and d^2/r^2, where d=least diameter.

	STEEL AND WROUGHT IRON														CAST IRON							
l/d	d^2/r^2														d^2/r^2							
	6	7	8	9	10	12	14	16	18	20	24	28	32	36	6	7	8	9	10	12	14	16
12	.903	.888	.874	.860	.847	.822	.799	.776	.755	.735	.699	.665	.633	.606	.537	.498	.466	.435	.410	.366	.331	.302
13	.888	.870	.855	.841	.826	.797	.772	.747	.724	.703	.663	.628	.596	.568	.498	.458	.426	.397	.372	.330	.297	.269
14	.872	.854	.836	.820	.803	.773	.745	.718	.693	.671	.630	.593	.560	.531	.459	.422	.389	.362	.338	.299	.268	.242
15	.856	.835	.816	.797	.781	.748	.717	.690	.663	.640	.597	.559	.526	.497	.426	.388	.357	.330	.308	.271	.241	.217
16	.839	.817	.796	.776	.758	.723	.691	.661	.634	.610	.566	.527	.494	.464	.394	.358	.328	.303	.281	.246	.218	.196
17	.822	.798	.776	.754	.735	.698	.664	.634	.606	.580	.535	.497	.464	.435	.366	.331	.302	.277	.257	.224	.198	.178
18	.804	.779	.755	.732	.712	.673	.638	.607	.578	.552	.507	.468	.435	.407	.340	.306	.278	.255	.236	.204	.181	.162
19	.786	.759	.735	.711	.689	.648	.613	.580	.552	.525	.480	.442	.409	.381	.316	.283	.257	.235	.217	.187	.165	.148
20	.769	.741	.714	.689	.666	.625	.588	.556	.526	.500	.454	.417	.384	.357	.294	.263	.238	.217	.200	.173	.151	.135
21	.751	.721	.694	.668	.644	.602	.564	.531	.502	.475	.430	.393	.362	.335	.274	.244	.221	.201	.184	.159	.139	.124
22	.732	.702	.674	.647	.623	.579	.542	.508	.478	.453	.408	.371	.341	.314	.257	.228	.205	.187	.171	.147	.129	.114
23	.716	.683	.654	.627	.602	.558	.519	.486	.456	.430	.387	.351	.319	.296	.240	.213	.191	.174	.159	.136	.119	.106
24	.698	.665	.634	.607	.581	.536	.498	.464	.436	.410	.367	.332	.303	.278	.224	.199	.178	.162	.148	.127	.110	.098
25	.680	.646	.615	.587	.561	.515	.478	.444	.416	.390	.348	.313	.286	.262	.211	.186	.167	.151	.138	.118	.103	.091
26	.663	.628	.596	.568	.542	.496	.458	.425	.397	.372	.330	.297	.270	.247	.198	.175	.156	.141	.129	.110	.096	.085
27	.646	.610	.578	.549	.523	.477	.439	.404	.379	.354	.314	.282	.255	.234	.186	.164	.146	.132	.120	.102	.089	.079
28	.630	.593	.560	.531	.505	.459	.422	.389	.362	.338	.298	.267	.242	.221	.175	.154	.138	.124	.113	.096	.084	.074
29	.613	.576	.543	.514	.487	.442	.404	.373	.346	.323	.284	.254	.229	.209	.165	.145	.129	.117	.106	.091	.078	.069
30	.596	.558	.526	.496	.469	.424	.387	.356	.330	.307	.269	.240	.216	.197	.156	.137	.122	.110	.100	.085	.073	.065
32	.565	.527	.494	.464	.438	.394	.358	.328	.302	.281	.245	.218	.196	.178	.140	.123	.109	.098	.089	.075	.065	.057
34	.535	.497	.464	.435	.409	.366	.330	.302	.278	.257	.224	.198	.178	.161	.126	.109	.098	.088	.080	.067	.058	.051
36	.507	.468	.435	.407	.382	.339	.306	.278	.255	.236	.204	.180	.162	.146	.114	.099	.088	.078	.072	.060	.052	.046
38	.480	.442	.409	.381	.356	.316	.283	.257	.235	.217	.188	.165	.147	.133	.103	.090	.080	.072	.065	.055	.047	.041
40	.454	.417	.385	.357	.333	.294	.263	.238	.217	.200	.172	.152	.135	.122	.094	.082	.072	.065	.059	.049	.043	.038
42	.430	.393	.362	.335	.312	.274	.245	.221	.201	.185	.157	.140	.124	.112								
44	.408	.371	.341	.314	.292	.256	.228	.205	.187	.171	.147	.129	.114	.103								
46	.386	.351	.321	.296	.274	.240	.213	.191	.174	.159	.136	.119	.106	.095								
48	.367	.331	.303	.278	.258	.224	.199	.178	.162	.148	.126	.110	.098	.088								
52	.330	.297	.270	.247	.228	.198	.174	.156	.141	.129	.110	.096	.085	.076								
56	.298	.267	.242	.221	.203	.175	.154	.138	.124	.113	.096	.084	.073	.065								
60	.270	.241	.217	.198	.182	.156	.137	.122	.110	.100	.084	.074	.065	.058								

COLUMN FORMULAS

For Steel and Wrought Iron

$$p=\frac{a}{1+\frac{l^2}{8000\,r^2}}$$

For Cast Iron

$$p=\frac{a}{1+\frac{l^2}{1000\,r^2}}$$

AVERAGE VALUES OF d^2/r^2		FOR LIGHT WEIGHT	FOR HEAVY WEIGHT
Even-legged Angles		26.0	26.4
Uneven " "		10.0	10.0
" " "		11.7	11.7
Channels	2 to 12-in.	6.8	6.8
"	15-in.	7.4	
"	2 to 15-in.	12.7	12.7
Deck-Beams	4 to 10-in.	7.1	7.1
" "	" " " "	35.0	35.0
I-Beams	6 to 15-in.	6.4	6.4
" "	" " " "	21.6	22.5

AVERAGE VALUES OF d^2/r^2		FOR LIGHT WEIGHT	FOR HEAVY WEIGHT
Solid Cylinder		16.0	
Hollow "	Thick. from 0.01 to 0.02 Diam.	9.0	
Solid Square		12.0	
Hollow "	Thick. from 0 01 to 0.12 Diam.	6.8	
Solid Octagon		15.3	
Hollow "	Thick. from 0.05 to 0.080 Diam.	8.9	
Built Struts Av.Sec. $b=D$		10.4	
" " " "		10.2	
Channel Chord		6.5	8.0
Built "		7.0	8.0

Select the proper value of l/d in the left-hand column of the table, and follow a horizontal line from this until it reaches a vertical column, at the top of which is the proper value of d^2/r^2, and there will be found, at the intersection, a percentage value which is to be multiplied by the following constants (a) to obtain the allowable compressive strains in pounds per square inch:

16k. FOR WROUGHT IRON, STATIC.
20k. " CAST " "
20k. FOR MEDIUM STEEL, STATIC.
30k. FOR NICKEL STEEL, STATIC.

ALLOWABLE COMPRESSIVE STRAINS FOR TIMBER COLUMNS.

This table gives the percentages of compressive strength for different values of $\frac{l}{d}$, and the corresponding allowable strains per square inch of cross-section for different timbers.

$\frac{l}{d}$	Percentage.	White oak.	White pine.	Georgia pine.
15.........	81.6	1.31 k.	0.98 k.	1.47 k.
16........	79.6	1.27 k.	0.96 k.	1.43 k.
17.........	77.5	1.24 k.	0.93 k.	1.40 k.
18.........	75.5	1.20 k.	0.91 k.	1.36 k.
19.........	73.5	1.18 k.	0.88 k.	1.32 k.
20.........	71.4	1.14 k.	0.86 k.	1.29 k.
21.........	69.4	1.11 k.	0.83 k.	1.25 k.
22.........	67.4	1.08 k.	0.81 k.	1.21 k.
23.........	65.4	1.05 k.	0.78 k.	1.18 k.
24.........	63.5	1.02 k.	0.76 k.	1.14 k.
25.........	61.5	0.99 k.	0.74 k.	1.11 k.
26.........	59.7	0.96 k.	0.72 k.	1.07 k.
27.........	57.8	0.93 k.	0.69 k.	1.04 k.
28.........	56.0	0.90 k.	0.67 k.	1.01 k.
29.........	54.2	0.87 k.	0.65 k.	0.98 k.
30.........	52.6	0.84 k.	0.63 k.	0.95 k.
32.........	49.4	0.79 k.	0.59 k.	0.89 k.
34.........	46.4	0.74 k.	0.56 k.	0.83 k.
36.........	43.6	0.70 k.	0.52 k.	0.79 k.
38.........	40.9	0.65 k.	0.49 k.	0.74 k.
40.........	38.5	0.62 k.	0.46 k.	0.69 k.
45.........	33.1	0.53 k.	0.40 k.	0.60 k.
50....... ..	28.6	0.46 k.	0.34 k.	0.52 k.
55.........	24.8	0.40 k.	0.30 k.	0.45 k.
60.........	21.7	0.35 k.	0.26 k.	0.40 k.
Column Formulas............		$\dfrac{1.6 \text{ k.}}{1+\dfrac{l^2}{1\,000\,d^2}}$	$\dfrac{1.2 \text{ k.}}{1+\dfrac{l^2}{1\,000\,d^2}}$	$\dfrac{1.8 \text{ k.}}{1+\dfrac{l^2}{1\,000\,d^2}}$

l = Length of column, in inches;
d = Least diameter of column, in inches.

Percentage of Increase to Live-Load Strains to Produce Static Equivalent, According to the Formula:

$$S = 125 - \frac{1}{8}\sqrt{2\,000\,L - L^2}$$

Where,

S = Increase, in percentage;

L = Length, in feet, of applied loading which produces maximum strain in the member.

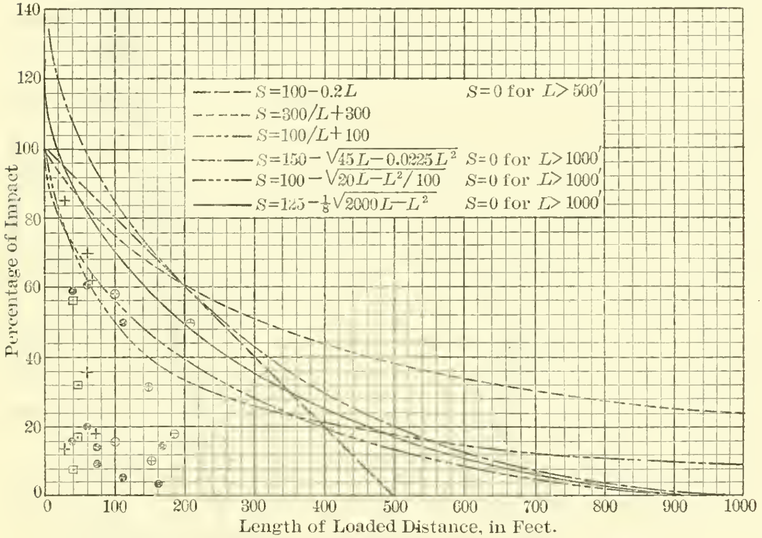
Applied loads shall be increased according to their various lengths by the above formula to produce the Static Equivalent.

L.	*S.*	*L.*	*S.*	*L.*	*S.*	*L.*	*S.*
1	119.41	25	97.22	150	59.15	390	25.95
2	117.10	26	96.68	160	57.18	400	25.00
3	115.32	27	96.15	170	55.28	410	24.07
4	113.83	28	95.63	180	53.45	420	23.17
5	112.52	29	95.12	190	51.70	430	22.29
6	111.33	30	94.61	200	50.00	440	21.44
7	110.23	35	92.22	210	48.36	450	20.60
8	109.22	40	90.00	220	46.78	460	19.79
9	108.27	45	87.91	230	45.24	470	19.00
10	107.37	50	85.97	240	43.76	480	18.23
11	106.51	55	84.12	250	42.32	490	17.48
12	105.69	60	82.35	260	40.92	500	16.75
13	104.91	65	80.67	270	39.57	510	16.03
14	104.16	70	79.06	280	38.25	520	15.34
15	103.43	75	77.51	290	36.97	540	14.01
16	102.73	80	76.01	300	35.73	560	12.75
17	102.05	85	74.57	310	34.52	580	11.56
18	101.39	90	73.17	320	33.35	600	10.44
19	100.75	95	71.82	330	32.20	650	7.91
20	100.13	100	70.51	340	31.09	700	5.76
21	99.52	110	68.00	350	30.01	750	3.97
22	98.92	120	65.63	360	28.95	800	2.52
23	98.34	130	63.37	370	27.93	850	1.41
24	97.78	140	61.21	380	26.92	900	0.63

Where L = 1 000 ft. or more, S = 0.

DIAGRAM OF IMPACT FORMULAS
AND
TESTS MADE ON BRIDGES OF THE BALTIMORE AND OHIO RAILROAD.

(For Mr. Greiner's tests see *Proceedings*, Am. Ry. E. and M. of W. Assoc., Vol. 6, 1905.)



+ Plate Girders.

⊕ Bottom Chord of Trusses.

● Main and Counter-ties of Trusses.

⊡ Hangers.

Vibrations were not noticeable at speeds of less than 15 or 20 miles per hour.

FIG. 7

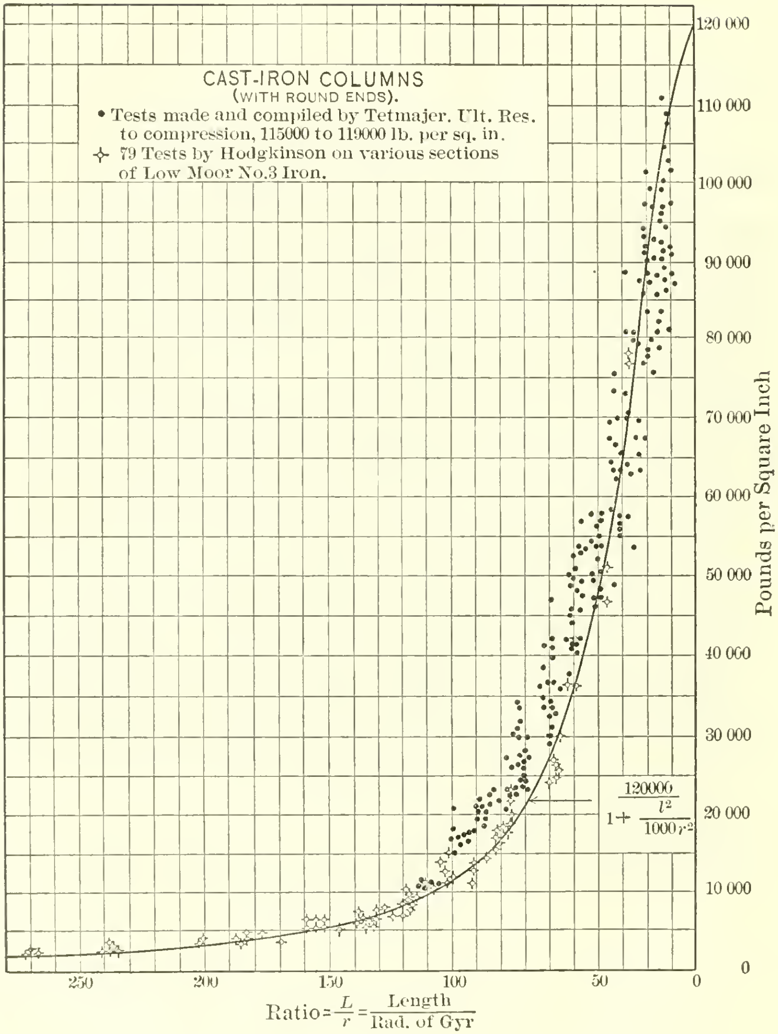
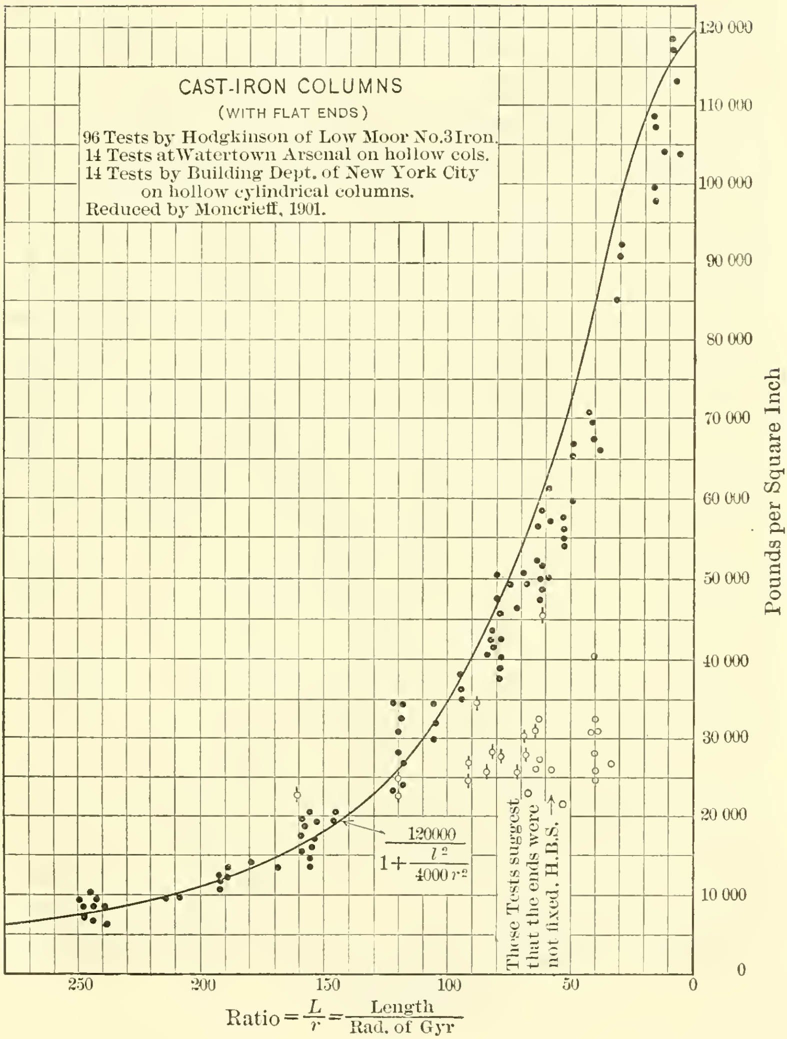

CAST-IRON COLUMNS
(WITH ROUND ENDS).
• Tests made and compiled by Tetmajer. Ult. Res. to compression, 115000 to 119000 lb. per sq. in.
79 Tests by Hodgkinson on various sections of Low Moor No.3 Iron.
120 000
110 000
100 000
90 000
80 000
70 000
60 000
50 000
40 000
30 000
20 000
10 000
0
Pounds per Square Inch
$\frac{120000}{1+\frac{l^2}{1000r^2}}$
250
200
150
100
50
0
Ratio = $\frac{L}{r}$ = $\frac{\text{Length}}{\text{Rad. of Gyr}}$

Fig. 8.



CAST-IRON COLUMNS
(WITH FLAT ENDS)
96 Tests by Hodgkinson of Low Moor No.3 Iron.
14 Tests at Watertown Arsenal on hollow cols.
14 Tests by Building Dept. of New York City on hollow cylindrical columns.
Reduced by Moncrieff, 1901.
120 000
110 000
100 000
90 000
80 000
70 000
60 000
50 000
40 000
30 000
20 000
10 000
0
Pounds per Square Inch
$\dfrac{120000}{1+\dfrac{l^2}{4000r^2}}$
These Tests suggest that the ends were not fixed. H.B.S.
250
200
150
100
50
0
Ratio $= \dfrac{L}{r} = \dfrac{\text{Length}}{\text{Rad. of Gyr}}$

FIG. 9.

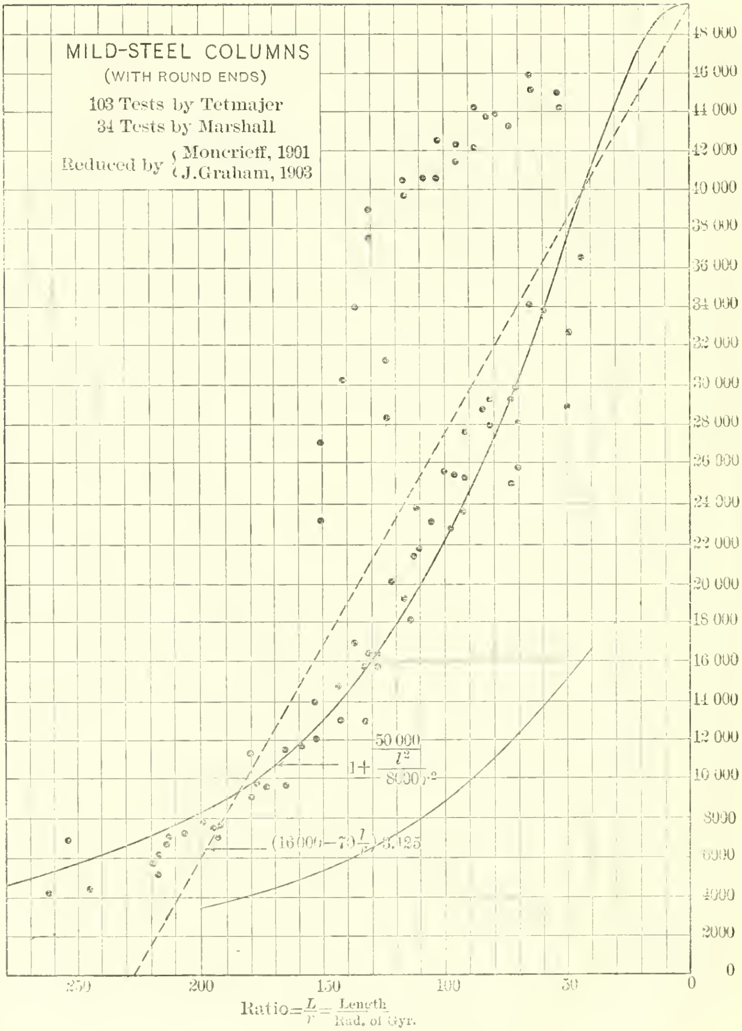
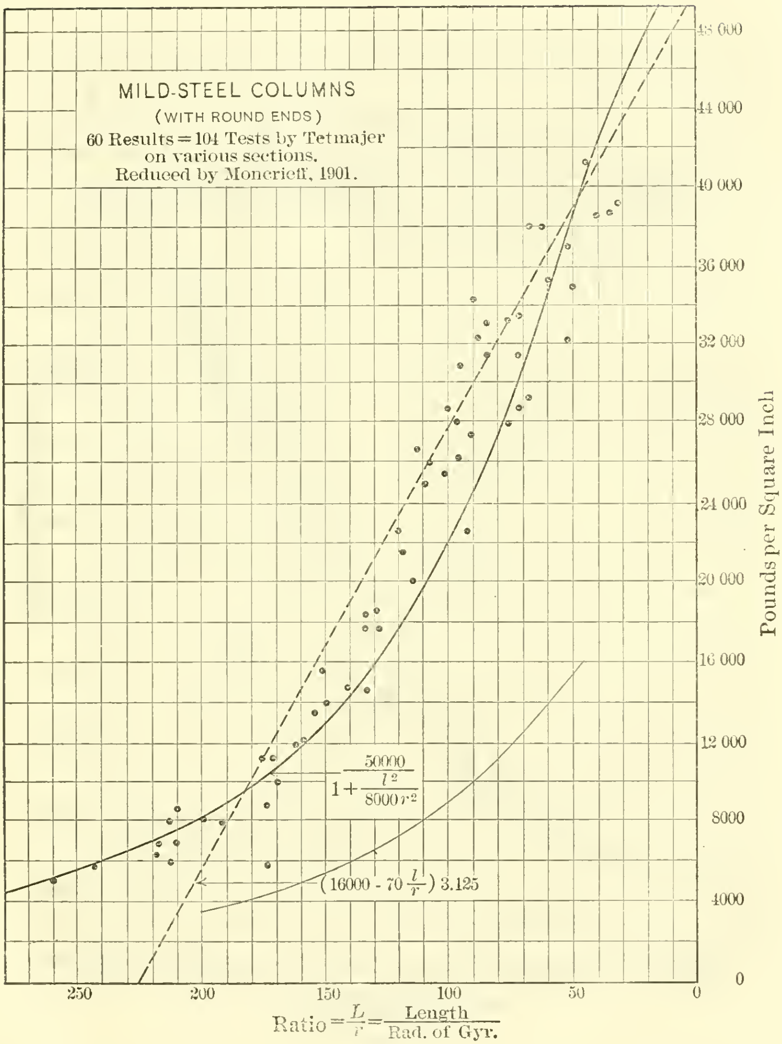

MILD-STEEL COLUMNS
(WITH ROUND ENDS)
103 Tests by Tetmajer
34 Tests by Marshall
Reduced by { Moncrieff, 1901 / J. Graham, 1903
$\dfrac{50\,000}{1+\dfrac{l^2}{8000\,r^2}}$
$(16000-70\frac{l}{r})3.125$
48 000
46 000
44 000
42 000
40 000
38 000
36 000
34 000
32 000
30 000
28 000
26 000
24 000
22 000
20 000
18 000
16 000
14 000
12 000
10 000
8000
6000
4000
2000
0
Pounds per Square Inch
250
200
150
100
50
0
Ratio $=\frac{L}{r}=\frac{\text{Length}}{\text{Rad. of Gyr.}}$

FIG. 10.



MILD-STEEL COLUMNS
(WITH ROUND ENDS)
60 Results = 104 Tests by Tetmajer on various sections.
Reduced by Moncrieff, 1901.
$\dfrac{50000}{1+\dfrac{l^2}{8000 r^2}}$
$(16000 - 70\frac{l}{r})$ 3.125
48 000
44 000
40 000
36 000
32 000
28 000
24 000
20 000
16 000
12 000
8000
4000
0
Pounds per Square Inch
250
200
150
100
50
0
Ratio $= \frac{L}{r} = \frac{\text{Length}}{\text{Rad. of Gyr.}}$

Fig. 11.

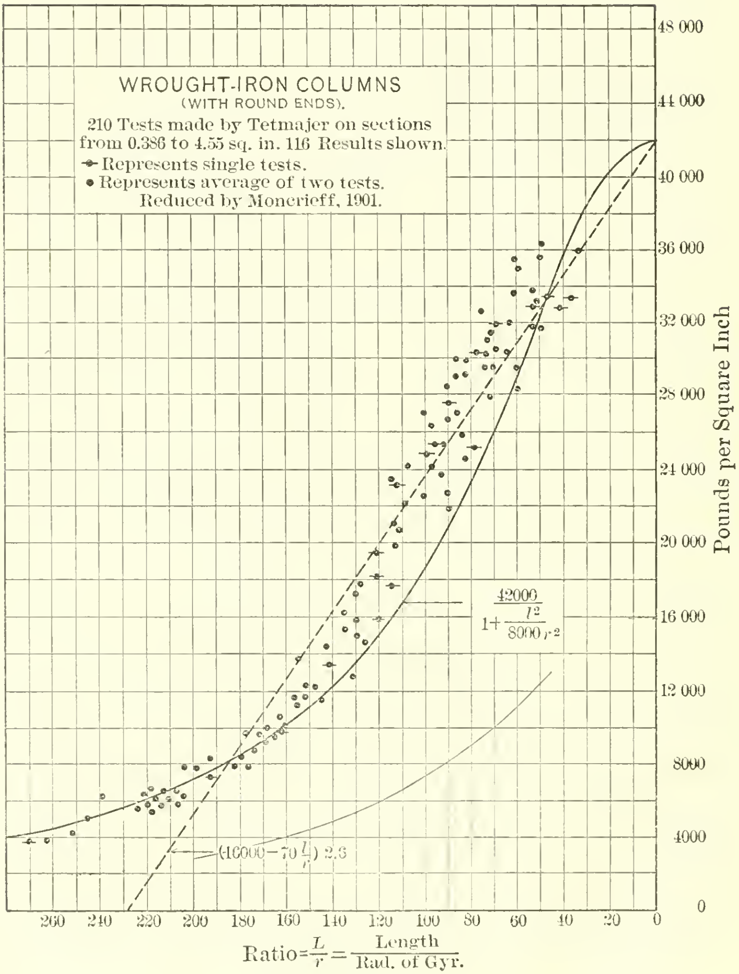

WROUGHT-IRON COLUMNS
(WITH ROUND ENDS).
210 Tests made by Tetmajer on sections from 0.386 to 4.55 sq. in. 116 Results shown.
Represents single tests.
Represents average of two tests.
Reduced by Moncrieff, 1901.
$\frac{42000}{1+\frac{l^2}{8000\,r^2}}$
$(46000-70\frac{l}{r})\ 2.6$
48 000
44 000
40 000
36 000
32 000
28 000
24 000
20 000
16 000
12 000
8000
4000
0
Pounds per Square Inch
260
240
220
200
180
160
140
120
100
80
60
40
20
0
Ratio $=\frac{L}{r}=\frac{\text{Length}}{\text{Rad. of Gyr.}}$

FIG. 12.

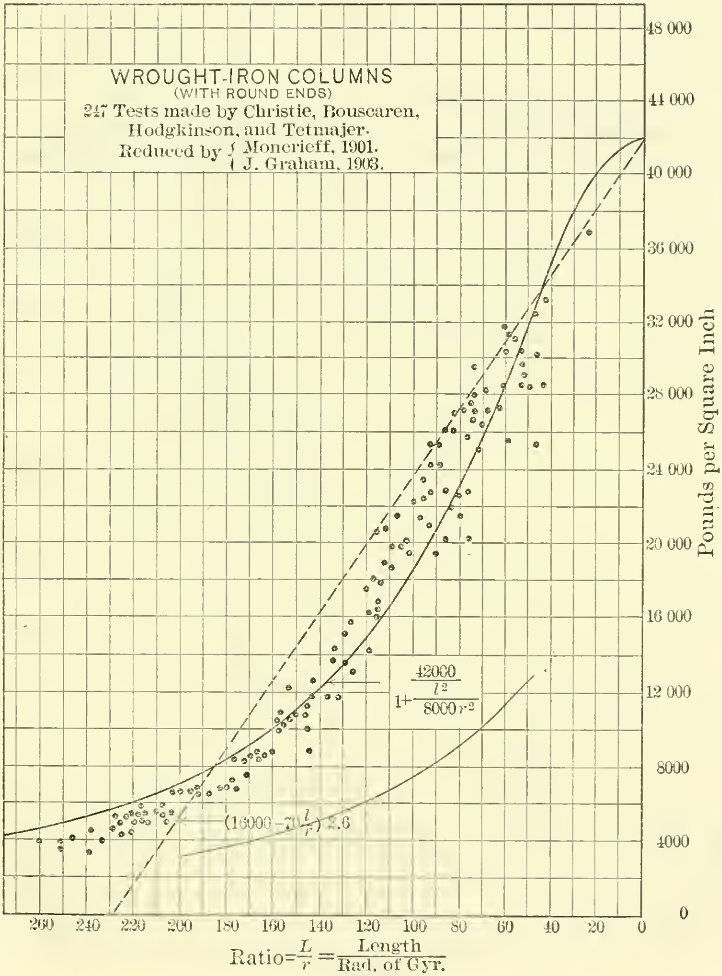

WROUGHT-IRON COLUMNS
(WITH ROUND ENDS)
247 Tests made by Christie, Bouscaren, Hodgkinson, and Tetmajer.
Reduced by { Moncrieff, 1901. J. Graham, 1903.
$\dfrac{42000}{1+\dfrac{l^2}{8000r^2}}$
$(16000-70\frac{l}{r})\,2.6$
48 000
44 000
40 000
36 000
32 000
28 000
24 000
20 000
16 000
12 000
8000
4000
0
Pounds per Square Inch
260 240 220 200 180 160 140 120 100 80 60 40 20 0
Ratio$=\dfrac{L}{r}=\dfrac{\text{Length}}{\text{Rad. of Gyr.}}$

FIG. 13.

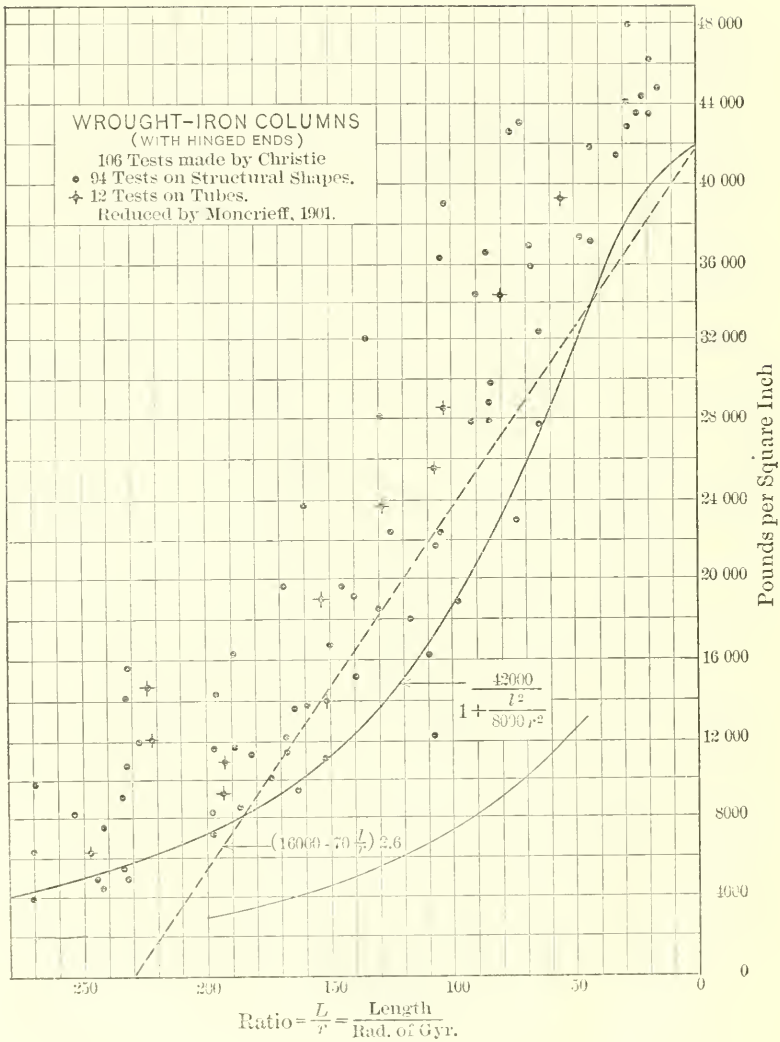

WROUGHT-IRON COLUMNS
(WITH HINGED ENDS)
106 Tests made by Christie
94 Tests on Structural Shapes.
12 Tests on Tubes.
Reduced by Moncrieff, 1901.
48 000
44 000
40 000
36 000
32 000
28 000
24 000
20 000
16 000
12 000
8000
4000
0
Pounds per Square Inch
250
200
150
100
50
0
$\frac{42000}{1+\frac{l^2}{8000 r^2}}$
$(16000 - 70\frac{l}{r})\,2.6$
Ratio = $\frac{L}{r} = \frac{\text{Length}}{\text{Rad. of Gyr.}}$

FIG. 14.

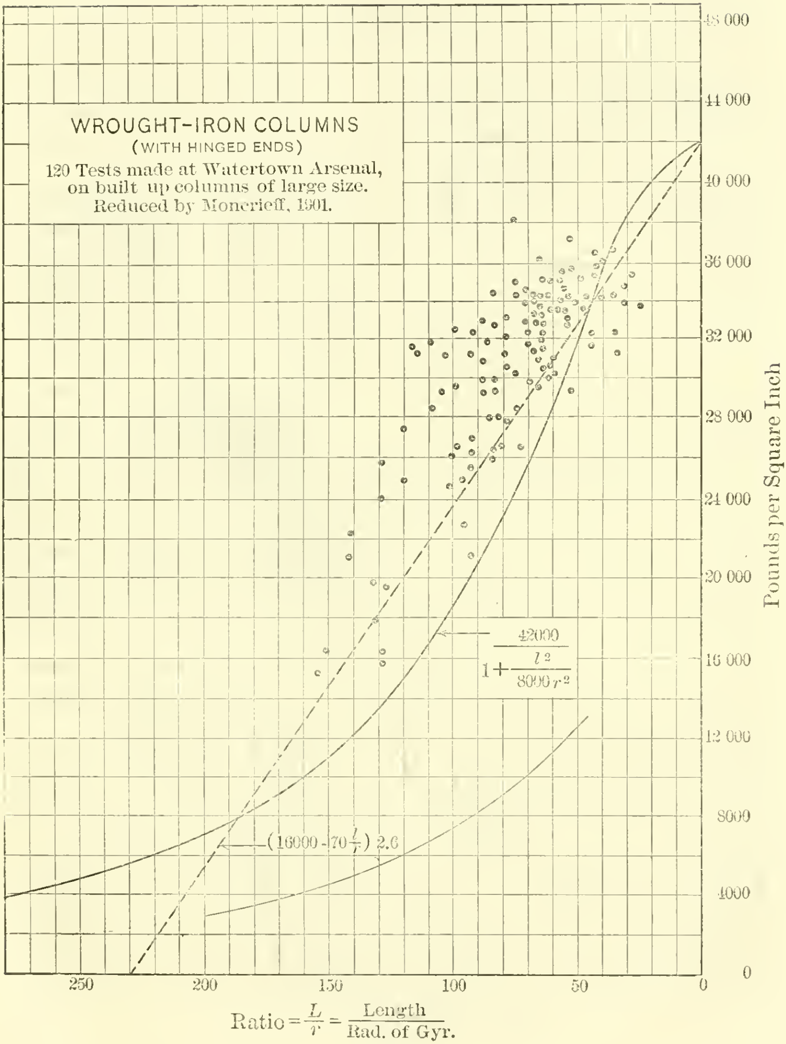
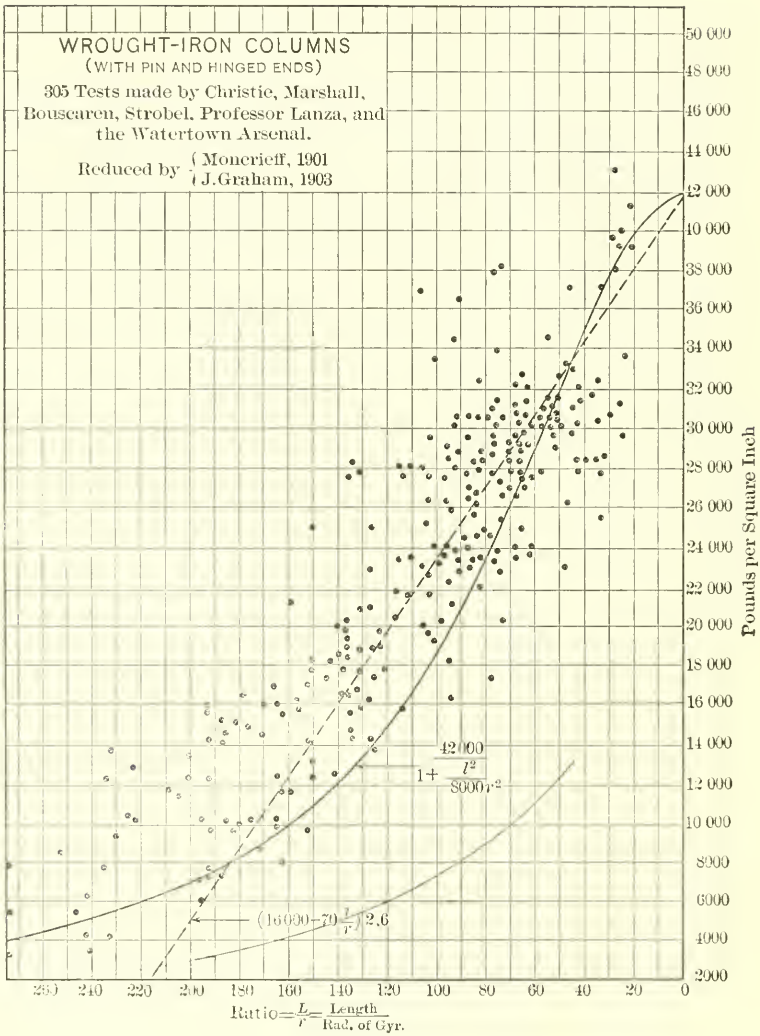

WROUGHT-IRON COLUMNS
(WITH HINGED ENDS)
120 Tests made at Watertown Arsenal,
on built up columns of large size.
Reduced by Moncrieff, 1901.
$\dfrac{42000}{1+\dfrac{l^2}{8000\,r^2}}$
$\left(16000-70\frac{l}{r}\right)2.6$
48 000
44 000
40 000
36 000
32 000
28 000
24 000
20 000
16 000
12 000
8000
4000
0
Pounds per Square Inch
250
200
150
100
50
0
Ratio $= \dfrac{L}{r} = \dfrac{\text{Length}}{\text{Rad. of Gyr.}}$

FIG. 15.



WROUGHT-IRON COLUMNS
(WITH PIN AND HINGED ENDS)
305 Tests made by Christie, Marshall, Bouscaren, Strobel, Professor Lanza, and the Watertown Arsenal.
Reduced by { Moncrieff, 1901 / J. Graham, 1903
$\frac{42000}{1+\frac{l^2}{8000r^2}}$
$(16000-70\frac{l}{r})2.6$
Pounds per Square Inch
50 000
48 000
46 000
44 000
42 000
40 000
38 000
36 000
34 000
32 000
30 000
28 000
26 000
24 000
22 000
20 000
18 000
16 000
14 000
12 000
10 000
8000
6000
4000
2000
240
220
200
180
160
140
120
100
80
60
40
20
0
Ratio $=\frac{L}{r}=\frac{\text{Length}}{\text{Rad. of Gyr.}}$

FIG. 16.

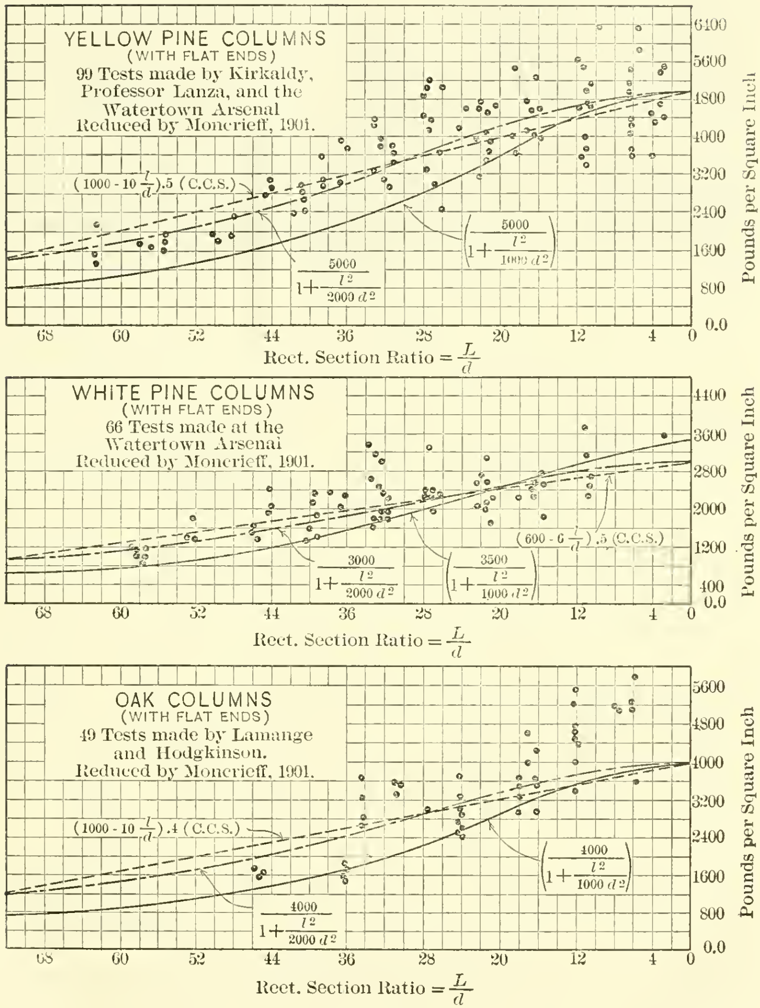

YELLOW PINE COLUMNS
(WITH FLAT ENDS)
99 Tests made by Kirkaldy, Professor Lanza, and the Watertown Arsenal
Reduced by Moncrieff, 1901.
$(1000 - 10\frac{l}{d}).5$ (C.C.S.)
$\frac{5000}{1+\frac{l^2}{2000\,d^2}}$
$\left(\frac{5000}{1+\frac{l^2}{1000\,d^2}}\right)$
Pounds per Square Inch
Rect. Section Ratio $= \frac{L}{d}$
WHITE PINE COLUMNS
(WITH FLAT ENDS)
66 Tests made at the Watertown Arsenal
Reduced by Moncrieff, 1901.
$\frac{3000}{1+\frac{l^2}{2000\,d^2}}$
$\left(\frac{3500}{1+\frac{l^2}{1000\,d^2}}\right)$
$(600 - 6\frac{l}{d}).5$ (C.C.S.)
Pounds per Square Inch
Rect. Section Ratio $= \frac{L}{d}$
OAK COLUMNS
(WITH FLAT ENDS)
49 Tests made by Lamange and Hodgkinson.
Reduced by Moncrieff, 1901.
$(1000 - 10\frac{l}{d}).4$ (C.C.S.)
$\frac{4000}{1+\frac{l^2}{2000\,d^2}}$
$\left(\frac{4000}{1+\frac{l^2}{1000\,d^2}}\right)$
Pounds per Square Inch
Rect. Section Ratio $= \frac{L}{d}$

FIG. 17.

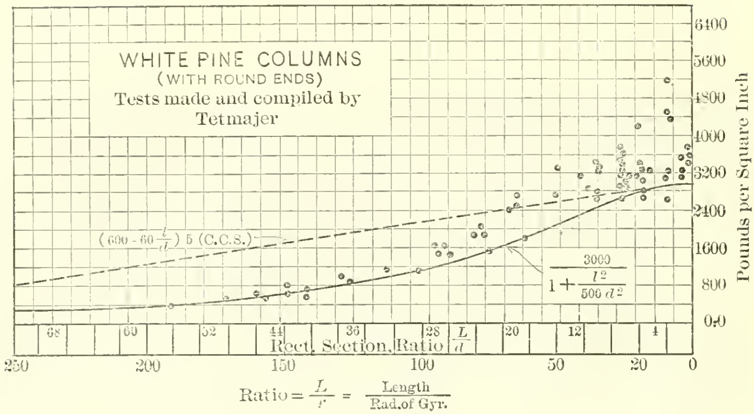

WHITE PINE COLUMNS
(WITH ROUND ENDS)
Tests made and compiled by
Tetmajer
$(600 - 60\frac{l}{d})$ 5 (C.C.S.)
$\frac{3000}{1+\frac{l^2}{500\,d^2}}$
6400
5600
4800
4000
3200
2400
1600
800
0.0
Pounds per Square Inch
68
60
52
44
36
28
20
12
4
Rect. Section, Ratio $\frac{L}{d}$
250
200
150
100
50
20
0
Ratio $= \frac{L}{r} = \frac{\text{Length}}{\text{Rad.of Gyr.}}$

FIG. 18.

DISCUSSION

S. W. Bowen, Assoc. M. Am. Soc. C. E. (by letter).—This very complete set of specifications is of much interest to the Profession in general, and especially to those engaged in designing and constructing bridges. Specifications such as these, which apply without modification to bridges of any span, are simple of application and a great help to the designer. Mr. Bowen.

The writer has long felt that the method of proportioning bridge members by using a constant unit stress and an allowance for impact is the logical one, and, for some time, has been collecting all available data on impact tests. On Plate X the impact tests made by Mr. Greiner, and shown on Mr. Seaman's diagram, Fig. 7, have been plotted; and to these have been added the tests made by the late S. W. Robinson, M. Am. Soc. C. E.,* and by F. E. Turneaure, Assoc. M. Am. Soc. C. E.,† together with those made by the American Railway Engineering and Maintenance of Way Association.‡

The tests made by Professors Robinson and Turneaure, and some of those made by Mr. Greiner, were taken from a diagram published by Henry S. Prichard, M. Am. Soc. C. E.,§ in connection with an article entitled "The Proportioning of Steel Railway Bridge Members."

In all, the results of one hundred and thirty-eight tests are shown on Plate X. These are mainly on comparatively short spans, the longest recorded being that of the 440-ft., double-track bridge on the Burlington Road over the Missouri River at Bellefontaine, Mo. Tests on longer spans and on very short ones are needed to cover the field fully, and it is to be hoped that the Maintenance of Way Association will present the results of such tests at an early date.

On Plate X the writer has plotted the impact formula, $S = \frac{100}{1 + \frac{L^2}{20\,000}}$, given by the American Railway Engineering and Maintenance of Way Association, and Mr. Seaman's formula, $S = 125 - \frac{1}{8} \sqrt{2\,000\,L - L^2}$. The latter seems to fit the results of the extreme tests better than the former, especially for loaded lengths of 150 ft. and greater. The single point falling seriously outside of both curves is the one for $L = 80$ ft. and $S = 100\%$ (this test may be unreliable).

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* *Transactions*, Am. Soc. C. E., Vol. XVI, p. 42.

† *Transactions*, Am. Soc. C. E., Vol. XLI, p. 410.

‡ Bulletin No. 125, July, 1910.

§ *Engineering News*, September 19th, 1907.

Mr. Bowen.

It would seem safe and reasonable to make the impact allowance for highway structures not more than 50% of that for railway structures, with a further material reduction of the impact allowance on the highway portion of the live load in the case of combined highway and railway bridges.

The unit tensile stress of 20 000 lb. per sq. in. for medium steel under static load, seems high, in view of the fact that secondary stresses, natural increase in the live load, and deterioration will add very considerably to the axial stress, and, in some cases, may bring the total up close to the elastic limit of the metal. In the writer's opinion, 16 000 lb. per sq. in. in tension, and 16 000 lb., properly reduced for compression, with a maximum in compression of 14 000 lb. per sq. in., comes more nearly within the safe carrying capacity of this material.

In the specifications for reinforced concrete, the requirement that beams and slabs shall be calculated as simply supported, seems to be rather severe. This clause was probably framed to comply with the New York Building Laws, which, until very recently, were extremely conservative in this respect. Most building laws allow such parts to be calculated for a bending moment at the center of the span of from $\frac{WL}{12}$ to $\frac{WL}{10}$; steel being provided, of course, to take the negative bending moments over the supports. Where the foundation is good, this method of computation is warranted by the monolithic character of the construction.

Mr. Cochrane.

VICTOR H. COCHRANE, M. AM. SOC. C. E. (by letter).—This paper is timely because of the fact that many bridge designers are realizing the desirability of a more or less thoroughgoing revision of the specifications in general use.

The writer has recently been engaged in revising the specifications used by his firm for the design and construction of bridges, and has reached certain conclusions somewhat at variance with those of Mr. Seaman.

It is gratifying to note that the old practice of reducing the working stresses to provide for the live-load effects is being abandoned. It certainly leads to greater clearness and consistency to reduce all loads to the basis of an equivalent dead load and then to use the same working stresses throughout. The saving in labor effected by this method is not inconsiderable, as the author remarks.

The most rational and satisfactory method of design, therefore, would appear to be to divide all the uncertainties into two classes, proceeding as follows:

1st. Increase the live-load strains by an increment which will take care of the uncertainties due to the application of the

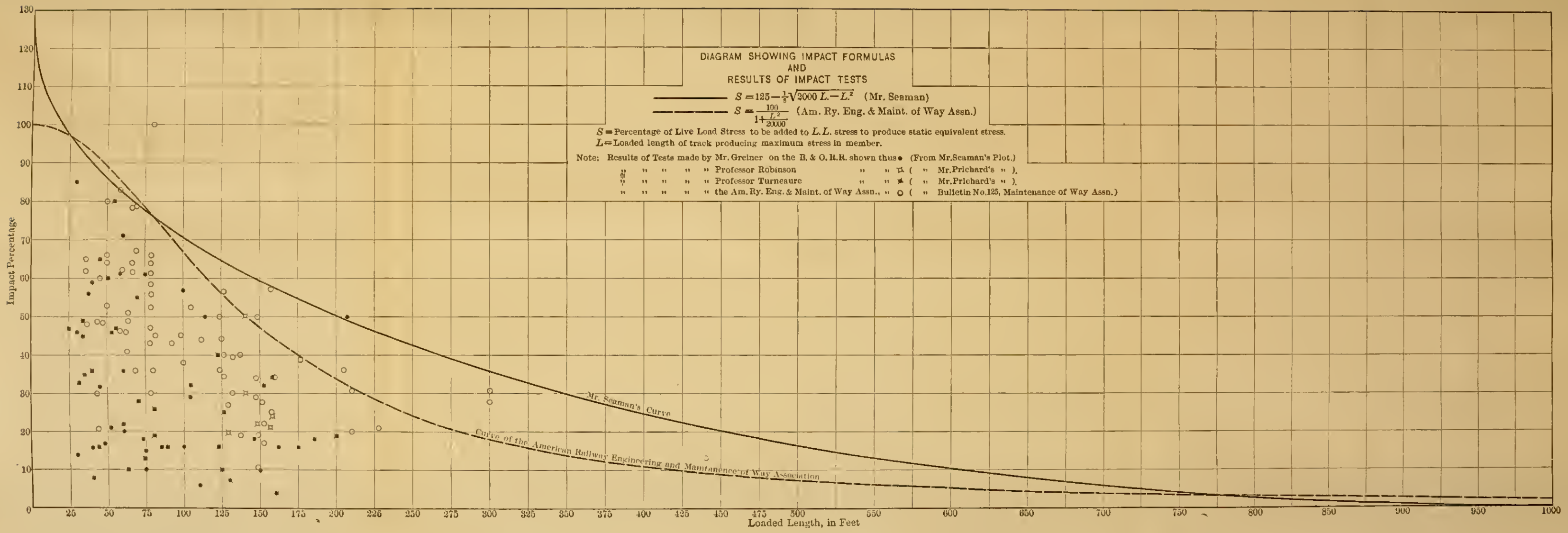
DIAGRAM SHOWING IMPACT FORMULAS
AND
RESULTS OF IMPACT TESTS
$S = 125 - \frac{1}{8}\sqrt{2000\,L. - L.^2}$ (Mr. Seaman)
$S = \frac{100}{1 + \frac{L^2}{20000}}$ (Am. Ry. Eng. & Maint. of Way Assn.)
S = Percentage of Live Load Stress to be added to $L.L.$ stress to produce static equivalent stress.
L = Loaded length of track producing maximum stress in member.
Note: Results of Tests made by Mr. Greiner on the B. & O. R.R. shown thus ● (From Mr. Seaman's Plot.)
" " " " " Professor Robinson " " (" Mr. Prichard's ").
" " " " " Professor Turneaure " " (" Mr. Prichard's ").
" " " " " the Am. Ry. Eng. & Maint. of Way Assn., " ○ (" Bulletin No. 125, Maintenance of Way Assn.)
Mr. Seaman's Curve
Curve of the American Railway Engineering and Maintenance of Way Association
Impact Percentage
Loaded Length, in Feet
0
10
20
30
40
50
60
70
80
90
100
110
120
130
25
50
75
100
125
150
175
200
225
250
275
300
325
350
375
400
425
450
475
500
550
600
650
700
750
800
850
900
950
1000

live load, that is, due to the fact that the load is moving instead of stationary. Mr. Cochrane.

2d. Use working stresses selected so as to provide for the uncertainties in regard to the behavior of the materials as used in the structure. Thus the working stresses should be chosen so as to take into account the effect of secondary stresses of various kinds, the variation in strength of compression members due to variation in the ratio $\frac{l}{r}$, and the like.

Mr. Seaman's static equivalent formula is intended to cover all the uncertainties of the first class. He seems to use the term "impact" in the sense of the effect due to the suddenness of application of the live load. The writer understands, however, that the term "impact" has long been used to mean precisely what the author calls "static equivalent." It would seem, however, that Mr. Seaman's term is a decided improvement.

The author does not seem to have taken into account the impact tests made under the direction of the American Railway Engineering and Maintenance of Way Association.*

In Fig. 19 are plotted some results of the tests by Mr. Greiner and those by the American Railway Engineering and Maintenance of Way Association. Only the maximum values are given, regardless of the type of structure or the character of the member tested. It should be remembered that these tests are all for railway loads. Fig. 19 also shows, for the sake of comparison, various impact or static equivalent formulas.

As brought out in Bulletin No. 125, just referred to, the chief cause of impact in long spans is the unbalanced condition of the locomotive drivers; and the maximum impacts occur when the time of rotation of these drivers is the same as the time of vibration of the span. In the case of short spans, such as plate-girder spans, the time of rotation of the drivers, even at the highest speeds, is greater than the vibration period of the span. In this case the impact values increase with the speed. Hence, there seems reason to believe that, from theoretical considerations, the impact may be less for very short plate-girder spans than for those somewhat longer. This view is borne out to some extent by the results of the tests. As will be seen in Fig. 19, the highest value of all (about 133%) occurred in the case of the 60-ft. span. Therefore, if the above statement be true, it may be found that some such curve as that shown by the line, C, for which the equation (using the author's notation) is $S = \frac{80}{100 + 1.1\,L - 12\sqrt{L}}$, represents the maximum impacts which might be obtained. The

* The results are published in Bulletin No. 125 of that Association.

Mr. Cochrane. values increase from 100% for the 5-ft. span to 119% for the 30-ft. span, and then decrease rapidly.

The writer believes, however, that this formula and Mr. Seaman's give higher values than necessary for the shorter spans. Values exceeding 100% are very exceptional, and it is thought that a maximum of 100% is quite sufficient.

The formula in most general use, $S = \frac{300}{L + 300}$, known as the American Bridge Company formula, is shown by the curve, B. Considering the lack of knowledge at the time it was put into use, it has served its purpose admirably. It appears, however, that it gives values somewhat too small for spans of 100 ft. and less, and much too large for spans of more than 200 or 300 ft. In the case of the 600-ft. span, the impact increment given by this formula is 33⅓%; while only 10% is given by the author's formula. Conservative designers will, perhaps, hesitate to make such a radical change in present practice. The writer has adopted the formula, $S = \frac{800}{800 + L + \left(\frac{L}{10}\right)^2}$, plotted as Curve D. It gives somewhat higher values than the author's formula for all spans exceeding about 40 ft.

The tests of the American Railway Engineering and Maintenance of Way Association seem to have established the fact that the ratio of dead load to live load has but little influence on the value of the impact increment. Consequently, the formula in general use, $S = \frac{L.\ L.}{L.\ L. + D.\ L.}$ (in which $L.\ L.$ denotes the live load and $D.\ L.$ denotes the dead load), cannot be justified. The curve, F, gives approximate values of this ratio for single-track railway spans designed for Cooper's E-50 loading. It will be seen that it gives values very much too high for long spans. The curve, E, is plotted from Waddell's formula, $S = \frac{400}{L + 500}$. The curve, G, gives the values determined by Melan's formula, $S = 0.14 + \frac{26}{L + 33}$. This formula is said to have been derived from theoretical considerations, but it is not at all in accord with experimental results. Some specifications in common use provide for impact by making the working stresses for dead load twice those for live load. This method is illogical, and leads to inconsistent results. Moreover, it leads to unnecessary difficulties in detailing connections, for the reason that these must be designed, not for computed, but for hypothetical stresses which will develop the full strength of the members.

Mr. Cochrane.

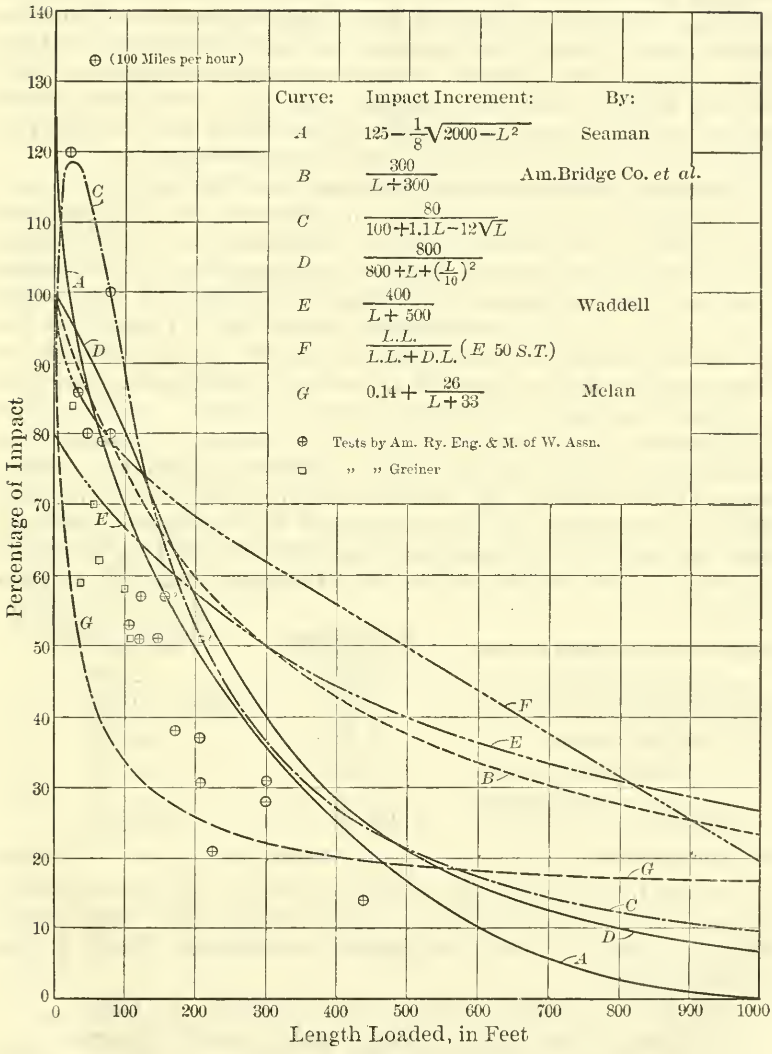

FIG. 19.

Mr. Cochrane.

Mr. Seaman has derived his impact formula from the results of tests on railway spans. It would seem to be an unwarranted procedure to apply this formula to highway loads. It is probable that the static equivalent for highway loads is only a small proportion of that for railway loads. Indeed, some specifications treat moving loads for highway bridges as static loads. The writer thinks it would be better to use a different formula for highway loads, or, in view of the present lack of knowledge concerning highway impacts, to use a certain proportion of the values for railway loads, say 33⅓ per cent.

Whatever impact increment is allowed must be considered in connection with the working stresses, in order to form a judgment as to the safety of the resulting design. Comparing the working stresses of the author's specifications with those in common use, it will be seen that the value for tension in medium-steel members is 25% higher than that in common use (16 000 lb. per sq. in.). Taking this fact in connection with the great reduction in the impact allowance for long spans, it will be seen that the author's specifications constitute a radical departure from current practice.

Relative to this matter, it may be of interest to calculate the bottom chord sections at the center of a 300-ft., single-track, railway span, by the author's specifications and by those of the American Railway Engineering and Maintenance of Way Association, and compare the results. The approximate equivalent live load for E-50 loading will be used for convenience, and the truss depth will be taken as 55 ft.

	By the author's specifications.	By the specifications of the Am. Ry. Eng. & M. of W. Ass'n.
Equivalent live load per linear foot per truss........	2 720	2 720
Impact increment..35.7% =	970	50% = 1 360
Dead load per truss........	1 350	1 800
Total load, W.......	5 040	5 880
Chord stress = $\dfrac{W \times (300)^2}{8 \times 55}$..	1 030 000	1 205 000
Section required...........	51.5 sq. in.	75.3 sq. in.

It will be seen from this calculation that in this case the Maintenance of Way specifications require sections about 46% greater than those obtained under the author's specifications. Hence it must be concluded:

1st. That the author's working stresses are much too high; or,

2d. That most specifications in use lead to wasteful designs.

Inspection of the values given by Mr. Seaman's static equivalent formula will show that they are quite small for what might be termed long spans. Then, granting (for the sake of argument) that short spans as designed under the specifications in general use are not

Mr. Cochrane.

heavier than it is advisable for safety to make them, it follows either that the author's working stresses are too high for long spans, or that higher working stresses may be used for long spans than for short ones. It may be argued that the latter alternative is justifiable on the ground that short spans are more likely to be overloaded than long ones. It would seem to be more logical, however, to provide for this tendency by specifying adequate live loads. There has been a well-defined disposition to use comparatively higher working stresses for long spans than for short ones. Probably this tendency has been due in some instances to the feeling that the specified live loads were excessive, but this statement could hardly be made in regard to the Quebec Bridge. The cantilever bridge at Beaver, Pa., recently completed, may be cited as an instance of a long-span bridge designed for heavy live loads and for working stresses substantially the same as those ordinarily used for short-span bridges.

Aside from the question whether the author's working stresses are, on the whole, too high, it is proper to inquire whether they are consistent among themselves.

A bearing value on pins of 30 000 lb. per sq. in. is very high. Relative to this matter, the paper by Mr. James E. Howard, "Some Tests of Large Steel Columns,"* furnishes valuable data, as was brought out clearly in the discussion by A. W. Carpenter and Charles Worthington, Members, Am. Soc. C. E. Long before the specified value of 30 000 lb. per sq. in. is reached the pin-plates will be permanently distorted at the pin-hole. For a column having the ratio, $\frac{l}{r} = 100$, the author's specifications would permit a unit bearing value on pins equal to nearly three and one-half times the working stress in the body of the member. Contrast this statement with the remark by Mr. Worthington, in his discussion of Mr. Howard's paper, that those tests "indicate that the working pressure on pins should not exceed the working compressive stress in the body of the member."

The consulting engineer, working under his own specifications, has the advantage that he does not have to consider the requirements thereof as fixed and immutable laws. He can modify the working stresses to suit the occasion. For example, in the case of a bridge having subdivided panels and carrying the floor loads directly on the bottom chords in bending, it would be proper, in view of the high secondary stresses in the chord due to the elongation of the hangers, to use low working stresses in the latter, so as to make the distortion small. It may be advisable to make some general distinctions in the specifications, such as to allow higher working stresses in stocky integral sections than in flimsy laced sections having the same $\frac{l}{r}$, and

* *Transactions*, Am. Soc. C. E., Vol. LXXIII, p. 429.

Mr. Cochrane. to require lower working stresses in the floor-beam hangers than in main members.

The specifications provide for braked train thrust as follows:

> "Provision shall be made for the sudden starting or stopping of a train 500 ft. in length, estimating the coefficient of sliding friction at 10 per cent."

This value is too small for short railway spans. The traction coefficient should decrease with the length loaded. The writer uses the following rules:

For lengths up to 80 ft. . . . . $T = 0.20$

For lengths of more than 80 ft. . . . . $T = \dfrac{20 + \dfrac{L}{20}}{40 + L}$

in which T = the coefficient of thrust,
and L = the length loaded.

As a matter of fact, there is, generally speaking, no such thing as the sliding of a train on the rails, because the brakes in use are not powerful enough to lock the wheels.

The allowable pressure on 1:2:4 concrete is given as 60 k. per sq. ft., or 416 lb. per sq. in. A value of 700 lb. per sq. in. would be more in keeping with the other working stresses.

In view of the high working stresses adopted, one might expect to find elaborate requirements for the purpose of insuring proper detailing, especially in the case of heavy compression members. In this respect the specifications are disappointing.

The provision that no allowance is to be made for spliced webs in calculating girder flanges is commendable, as is the requirement that the splices of abutting compression members must transmit 50% of the stress.

Mr. Frye. ALBERT I. FRYE, M. AM. SOC. C. E. (by letter).—There are two points in this timely and valuable paper which the writer will discuss, namely: (1) the weight per cubic foot of materials; and (2) the formula for reducing applied loads to their approximate static equivalents.

The weight of steel should be given as 489.6 lb. per cu. ft. (2% heavier than iron, at 480 lb. per cu. ft.). The weights of earth, timber, and masonry, should be tabulated more in detail, with varying values for the different classes.

The author gives the following formula for deriving the static equivalent of applied loads:

$$S = 125 - \frac{1}{8}\sqrt{2\,000\,L - L^2}.$$

Where, Mr. Frye.

S = Increase, in percentage;

and L = Length, in feet, of applied loading which produces maximum strain in the member.

The writer proposes the following, because it is much simpler, and, in his opinion, gives values of S more nearly correct for the shorter lengths of loading:

$$S = \frac{25\ 000 - 25\ L}{L + 200}.$$

For comparison, the values of S derived from these two formulas are given in Table 3.

TABLE 3.—COMPARATIVE VALUES OF S BY TWO FORMULAS.

L.	S. (Frye.)	S. (Seaman.)	L.	S. (Frye.)	S. (Seaman.)	L.	S. (Frye.)	S. (Seaman.)
0	*125.0	*125.0	90	78.5	73.2	300	35.0	35.7
10	117.9	107.4	100	75.0	70.5	350	29.5	30.0
20	111.4	100.1	120	68.7	65.6	400	*25.0	*25.0
30	105.4	94.6	140	63.2	61.2	500	17.9	16.8
40	100.0	90.0	150	60.7	59.2	600	12.5	10.4
50	95.0	86.0	160	58.3	57.2	700	8.3	5.8
60	90.4	82.4	180	53.9	53.5	800	5.0	2.5
70	86.1	79.1	200	*50.0	*50.0	900	2.3	0.6
80	82.1	76.0	250	41.7	42.3	1 000	*0.0	*0.0

* The two curves intersect at the four points, L = 0, 200, 400, and 1 000.

From Table 3 it will be noted that the increase in percentage, or "impact," is 100 for a 40-ft. length of loading, which is about the usual length (two panels) for maximum floor-beam reactions, and corresponds to what the writer considers good practice.

This is the reason for making the values of S, for short lengths of loading, greater than those given by the author's formula. The value, 25 L, in the numerator of the second term of the formula can be obtained mentally, of course, by multiplying L by 100 and dividing by 4. Formulas of this character, necessarily empirical, must be in the simplest form possible if they are expected to come into general use.

F. W. GARDINER, M. AM. SOC. C. E. (by letter).—In Mr. Seaman's specifications the following statement is found: Mr. Gardiner.

"Provision shall be made for the sudden starting or stopping of a train 500 ft. in length, estimating the coefficient of sliding friction at 10 per cent."

The force due to the starting or stopping of a train depends on the rate of acceleration or retardation. For trains of motor cars of the latest subway or suburban type, the motor equipment gives an acceleration having a maximum value of about $1\frac{1}{2}$ miles per hour per sec., and this rate is nearly uniform during the period of acceleration. The

Mr. Gardiner. effect of this rate is a longitudinal force on the track of $91.3 \times 1\frac{1}{2} = 136.9$ lb. per ton of train weight, or 6.8 per cent. The latest type of braking equipment developed for use on ten-car trains of the Interborough Rapid Transit Company of New York retards the train for service stops at a rate having a maximum value of about $2\frac{1}{2}$ miles per hour per sec., corresponding to a longitudinal force on the track of $91.3 \times 2\frac{1}{2} = 229.2$ lb. per ton of train weight, or 11.4 per cent. This braking equipment retards the train, for emergency application, at an average rate of about $2\frac{1}{2}$ miles per hour per sec., but the rate is not uniform, and at the instant of stopping suddenly rises to about 5 miles per hour per sec. For emergency application of the brake at the instant of stopping, the longitudinal force on the track, therefore, is $91.3 \times 5 = 456.5$ lb. per ton of train weight, or 22.8 per cent. Emergency application occurs frequently, especially on trains equipped with automatic trips, and structures over which equipment of this type is operated will frequently have to resist a longitudinal force of 23% of the weight of one train.

Mr. Fuller. ALMON H. FULLER, M. AM. SOC. C. E. (by letter).—The fact that the author has departed from the usual custom of allowing a higher unit stress for a combination of wind stresses with dead and live loads, than for those from dead and live loads alone, emphasizes the uncertainty which frequently exists in determining the proper unit stress for combinations of compression and bending such as occur occasionally, especially in the end posts of an ordinary bridge.

In light structures, it is not at all unusual that the actual unit stresses in the end post, due to transverse bending from the wind load and from eccentricity, exceed in magnitude those due to direct column action, while the allowable compressive stresses, reduced by the column formula, are only about one-half of those allowed for bending.

Although stresses from portal action are still occasionally neglected, under the assumption that the wind "shins down the post," the usual practice, as far as the writer is aware, is to base unit stresses for combined stresses in the end post on those indicated by the column formula. This practice, of course, is "on the safe side," but the possibility that it imposes unnecessary severity in many instances led the writer, about a year ago, to begin, in a modest manner, some combined compression and bending tests.

The only results available thus far are those from some small timber specimens, $1\frac{1}{2}$ by $1\frac{1}{2}$ in. by 2 ft., tested as thesis work by two seniors (Messrs. G. R. Edwards and C. A. Irle) at the University of Washington. These results indicate that, under the conditions under investigation, the actual stress at the elastic limit, and also at rupture, varies directly with the transverse load, from the values obtained in direct compression to those obtained in pure bending.

The writer realizes fully that these forty odd tests on small timber specimens cannot be used as a basis for writing a specification for the design of large steel members, yet he feels the need for something more specific than the existing specifications, and mentions the work already completed with the hope that it may possibly bring out further discussion on the subject. Mr. Fuller.

S. M. Swaab, M. Am. Soc. C. E. (by letter).—The live load which is transmitted through the earth from the street surface to the subway roof is of varying intensity, depending, in the first place, on the distribution of the load on the surface, secondly, on the number and arrangement of the underground structures, thirdly, on the depth of fill on the roof, and lastly, on the cohesion of the soil. This latter quantity is usually negligible, for, when the structure is built by cut-and-cover method, there is no cohesion to speak of in the back-filled earth, although it is possible that, as the soil becomes compacted, this condition may be developed. There is doubtless a greater percentage of the load transferred from the surface to the subway roof where the fill on the top of the structure is slight than where the depth is considerable, but what portion of the load is transferred, and in what manner it is tranferred, are the facts to be determined. The writer desires to know in what way it is proposed to transfer the local concentration of 100 k., on four wheels, 12 ft. between axles and 8 ft. gauge, to the subway roof, and also why a concentrated load without regard to any limitations as to area covered (as is stated in the proposed specification) may be considered as distributed over an area of 4 sq. ft. and thence through the earth on a slope of $\frac{1}{2}$ to 1. Mr. Swaab.

J. B. French, M. Am. Soc. C. E. (by letter).—These specifications are confined entirely to matters of design, and contain many valuable data, well arranged and co-ordinated for the designer's use. The author's avowed purpose has been to prepare a single set of general specifications, which shall apply not only to bridges for all classes of traffic, with spans from 16 to 1 600 ft., but also for rapid transit subways, and for all the various materials of construction entering into these structures. Mr. French.

For this purpose, the specified stresses for steelwork, which is most prominent in the specifications, are based on an "allowable static strain" of 20 000 lb. per sq. in. of net section, and all moving loads, regardless of their character or method of application, are increased according to the "static equivalent," or impact formula,

$$S = 125 - \frac{1}{8}\sqrt{2\,000\,L - L^2},$$

where L = length, in feet, of the applied loading which produces maximum stress in the member.

Mr. French.

The general discussion which followed a similar paper* presented by Mr. Seaman in 1899 undoubtedly had much to do with the uniformity which has been attained since that time in general specifications for steel structures. The advantages of this uniformity are obvious, but it is somewhat doubtful whether the greater generalization suggested in the present paper would be equally advantageous.

Before discussing this question, therefore, it seems pertinent to state briefly what has been accomplished in this direction during recent years.

By organized and intelligent co-operation on the part of engineers representing both manufacturers and purchasers, and others not attached to either group, in the Committee on Steel Structures of the American Railway Engineering Association, specifications for a single grade of structural steel have been prepared, which have since been adopted, with slight modifications, by the American Society for Testing Materials as its "Standard Specifications for Structural Steel for Bridges." The latter society has also adopted "Standard Specifications for Structural Steel for Buildings," defining another single but somewhat inferior grade of material.

Through the efforts of this same committee, the "General Specifications for Steel Railroad Bridges," incorporating the specifications for structural steel previously mentioned and including detailed requirements for design and workmanship, have also been adopted by the American Railway Engineering Association, in which the unit stresses are based on 16 000 lb. per sq. in. of net section for axial tension, and live-load stresses (from railroad traffic only) are increased by a "dynamic increment" determined by the commonly used impact formula, $I = S \frac{300}{L + 300}$, where L has the same meaning as in Mr. Seaman's formula.

These Railway Association specifications have been quite generally adopted, in whole or in part, by leading railroads, and the 16 000-lb. unit stress has also been adopted for highway bridges and for the steel-work of buildings by such authorities as G. F. Swain, M. Am. Soc. C. E., in the "Specifications for Bridges Carrying Electric Railways," written for the Massachusetts Railroad Commission, and by C. C. Schneider, Past-President, Am. Soc. C. E., in his specifications for "Structural Work of Buildings."†

In these previous efforts toward uniformity, the adopted standard grades of structural steel and an allowable unit stress of 16 000 lb. per sq. in. in axial tension, therefore, appear to be closely associated, but each has been confined to specifications for ordinary structures, the

* *Transactions*, Am. Soc. C. E., Vol. XLI, p. 140.

† *Transactions*, Am. Soc. C. E., Vol. LIV, p. 490.

steelwork for which is usually wanted quickly and at the ordinary market price. Mr. French.

It can be fairly claimed that rolled or fabricated material which complies fully with the foregoing specifications, undoubtedly represents the best average product of modern mills and shops, and that the purchaser of such material will get the quickest deliveries and the lowest prices.

It does not follow, however, that this standard material, which is manufactured to meet the average requirements for ordinary structures, is equally suitable for more particular and special purposes, such as the main members in very long bridges, in which the dead weight has to be reduced as much as possible; and, if special material is required, special rules of design and special requirements for workmanship and fabrication will necessarily be required also.

The writer thinks, therefore, that further generalization of specifications for structural steelwork should include the adoption of these standard grades of structural steel and of unit stresses based on 16 000 lb. per sq. in. for axial tension, and that they should be confined to the ordinary structures for which such material and stresses are suitable.

In regard to impact formulas: It may be of interest to note that the formula, $I = S \dfrac{300}{L + 300}$ used in the specifications of the American Railway Engineering Association, was first suggested by Fred. Thompson, M. Am. Soc. C. E., and was first published in the bridge specifications of the Southern Railway Company, issued in 1894, as a substitute for the formula, $I = S\left(0.10 + \dfrac{220}{L + 240}\right)$, used by Mr. Schneider in the revised Pencoyd specifications of the same year. On account of its greater simplicity, and the close similarity of the values given to those derived from the Pencoyd formula, Mr. Schneider substituted it for the latter in his specifications of 1895, and used it in his later specifications for the American Bridge Company. It is frequently referred to, therefore, as the "American Bridge Company" formula.

It should be kept in mind, however, that, when the Pencoyd specifications were published in 1887, and for years afterwards, data, derived from actual measurements on existing bridges under American conditions of track, rolling stock, and train speeds, were almost entirely lacking, as was stated by Mr. Schneider in his discussion* on the "Life of Railroad Bridges" in 1895; and, while logical scientific grounds and excellent reasons for using an impact formula and uni-

* *Transactions*, Am. Soc. C. E., Vol. XXXIV, pp. 328-335.

Mr. French. form working stresses were clearly stated in that discussion, no claim was made that the formula was scientifically accurate or final, as is shown by the following statement:

"If future experiments should prove that the coefficients for impact adopted are too large, they can be easily corrected, which, however, would not affect the system, but would only involve a change of the numerical values of the coefficients."

In 1887, and, to a diminishing extent, up to within the last ten years, solid ballasted concrete floors for railroad bridges were rarely used, and the wooden cross-ties delivered the live loads directly into the steel structure. The rails, ties, and guard timbers of the old type of floor, together with the steel stringers and floor-beams for ordinary single-track plate girders or truss spans, weighed less than 1 000 lb. per lin. ft. of track, while the rails, ties, ballast, concrete, and steel-work in the floor of the ordinary plate-girder spans with the solid ballasted floors, now commonly used in grade-crossing eliminations in cities, frequently have a total weight of more than 5 000 lb. per lin. ft. of track.

With any type of floor varying comparatively little in weight per foot for spans of different lengths, the effect of the suddenness of the application of the load to the girders or truss members can reasonably be considered as inversely proportional to the length of the structure or of its loaded portion. The weight, inertia, and cushioning effect of the construction or medium through which the live load is transmitted to the member, however, should certainly be taken into account in the determination of the proper increment to be added for impact or to produce the "static equivalent."

The most recent and most extensive series of impact tests on railroad bridges are those made by the sub-committee of the Committee on Steel Structures of the American Railway Engineering Association. These tests are described* very fully, and the following statements in the report of the sub-committee have a direct bearing on this discussion: On page 13, in speaking of the method of conducting the tests, the report states:

"Little difference was noted in the results at various speeds below 15 miles per hour, and, in general, the results at 10 miles per hour may be considered as practically equal to static stresses."

This seems to show that no increase is necessary to produce the static equivalent of stresses from such moving loads as pedestrians and vehicles passing over bridges or subways. On pages 32 and 33, under "Summary of Results," the following statements are made:

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* *Bulletin* No. 125, American Railway Engineering Association. July, 1910.

Mr. French.

"The maximum impact percentage, as determined by these tests, is closely given by the formula,

$$I = \frac{100}{1 + \frac{l^2}{20\,000}},$$

in which I = impact percentage and l = span length [and from which the maximum value for I is 100% when $l = 0$].

"The effect of differences of design was most noticeable with respect to differences in the bridge floors. An elastic floor, such as furnished by long ties supported on widely spaced stringers, or a ballasted floor, gave smoother curves than were obtained with more rigid floors. The results clearly indicated a cushioning effect with respect to impact due to open joints, rough wheels and similar causes. This cushioning effect was noticed on stringers, floor beams, hip verticals and short-span girders."

As bearing on this question of the cushioning effect of solid ballasted floors, the following statement, by the late G. H. Thomson, M. Am. Soc. C. E., in 1892, speaking of his experience on the New York Central Railroad, seems pertinent:*

"A considerable number of abutments and piers hammered out by deck plate bridges of small span, coming within the experience of the writer, will require an outlay to rebuild them equal to ten times the cost of new solid floor bridges; the injured masonry, however, will sustain the solid floor bridges, though not equal to the task of sustaining the old-fashioned plate bridges."

A. F. Robinson, M. Am. Soc. C. E., Bridge Engineer of the Atchison, Topeka and Santa Fé Railroad, in a paper† before the Western Society of Engineers in 1904, also stated that solid ballasted floors had been built under his direction to replace long timber ties resting on shelf-angles in half-through plate-girder spans, and that, notwithstanding the increased weight of the floor and the greater stresses, the deflection and vibration of the old steel girders were less than before the heavier floor had been put in.

On the strength of these statements and similar testimony, the writer, some years ago, adopted the following rule in the design of railroad bridges carrying such floors:

"In the case of main girders or trusses, and their supports, carrying fully ballasted track, one-half the dead-load stress shall be subtracted from the full live-load stress, before applying the impact formula."

In the design of the floor itself, he has treated the engine concentrations as uniformly distributed over a length equal to the distance between the axle centers and over a width of 10 ft., plus 100% for impact (that is, for E-50 loading, the floor is designed for a uni-

* *Transactions*, Am. Soc. C. E., Vol. XXVII, p. 507.

† *Journal*, Western Soc. of Engrs., Vol. X, No. 3, p. 253.

Mr. French. formly distributed load of 1 000 lb. per sq. ft., including impact). Particular care was taken, however, to have the concrete encase the steel floor-beams and lower girder flanges thoroughly, and to bond the concrete and the steelwork together as effectively as possible, for the purpose, not only of reducing impact and making the two materials work together, but also of deadening the noise; and the structures thus designed are giving a good account of themselves in service.

The only provision in Mr. Seaman's specifications relative to structural steelwork embedded in concrete is the following: "When beams and girders are embedded in concrete, the allowable flange strain may be increased 25 per cent."

This general type of construction is now used extensively, and occupies a distinct position of its own, intermediate between ordinary structural steelwork and reinforced concrete; and, in the writer's opinion, its design is worthy of careful and detailed attention. In the first place, such construction should be designed and built so that the concrete will protect the steelwork from corrosion permanently and effectively, and, if that is accomplished, the author's requirement in regard to "minimum section" can be properly revised to permit the use of ¼-in. material. In the second place, there seems to be no sufficient reason why this composite construction should not be designed so as to make the concrete work with the steel in resisting compressive stresses. They must, in fact, work together in this way, unless the bond between them is destroyed, and steel compression flanges designed by the author's rules can never receive the specified stresses unless the encasing concrete is crushed or slips on the steel.

One great advantage which this type has over ordinary reinforced concrete construction is illustrated in its use for bridges carrying streets over steam railroad tracks, where the maintenance of uninterrupted railroad traffic is essential. The structural steel frame can be erected at night or between trains, and is immediately available to support the forms and carry the weight of the plastic concrete; and, by the time it is necessary for the bridge to carry full live and dead loads, the concrete has set, forms a working part of the structure, and also serves as the most effective means yet devised to protect the steelwork from locomotive gases.

The writer prefers, therefore, as far as practicable, to treat a plate girder embedded in concrete exactly as he treats a so-called unit frame of rods or other steel reinforcement embedded in a reinforced concrete beam; in other words, to proportion the steel in the tension flanges for the maximum total load at the regular tensile unit stress (preferably 16 000 lb. per sq. in. of net section); to proportion the web-plate to carry all shearing stresses; and to proportion the steel in the compression flanges to carry, unaided, all stresses from initial loads which can occur before the concrete has set, and rely on the concrete, as a composite part of the beam, to resist all subsequent compressive stress. This practice, of course, requires particular care in order to insure an effective bond between the concrete and the steel, and makes careful workmanship and thorough inspection a necessity, but not to a greater degree than with reinforced concrete. Mr French.

The roof of a subway station with a fairly deep earth cover, where spans are too long for ordinary reinforced concrete slabs, also presents conditions admirably adapted to this method of design. The maximum initial load which the steel girder has to carry unaided, consists of the weight of the forms and the plastic concrete only, all additional load, both dead and live, being applied after the concrete has set.

A six-track railroad bridge of short, shallow spans, designed in this manner, and carrying both the heavy freight and high-speed passenger traffic of an Eastern trunk-line railroad, with only 6 in. of ballast under the ties, has been in service considerably more than a year, and is giving entire satisfaction.

In regard to the design of reinforced concrete: In Mr. Seaman's specifications, the maximum tension in the steel is fixed at 20 000 lb. per sq. in., the same as for structural steel, but it is stated that "from 0.70 to 1.50% of steel gives the usual economical sections." These percentages, coupled with a specified "maximum compression in concrete of beams" of 600 lb. per sq. in., a ratio $\frac{E_s}{E_c} = 15$, and straight-line deformation, mean that the corresponding unit tensile stress in the steel will vary from about 15 600 to 9 600 lb. per sq. in. In this case, also, the commonly used unit stress of 16 000 lb. per sq. in., therefore, would seem more suitable than the 20 000 lb. specified.

In the specifications for reinforced concrete, also, the clause, "Beams and slabs shall be calculated as simply supported, but rods shall be placed in compression flanges over supports, to prevent cracking," appears to put an undesirable check on the designer. Undoubtedly, this clause was written to fit the reinforced concrete cross-section so uniformly used in the New York subways, namely, what may be described as a box-section, the sides of which are designed as discontinuous single-span slabs, arbitrarily reinforced at corners and over supports sufficiently "to prevent cracking," no method being prescribed by which the size or length of such reinforcement may be determined.

The writer believes that the cross-section of a subway, like that of a tunnel or sewer, if built of reinforced concrete, should be designed to be as nearly structurally continuous as possible, and that the conditions governing the execution of such work are too varied to make the use of a uniform type of cross-section, or such a rule as the one quoted, desirable in a set of general specifications.

Mr. French.

On the other hand, rules of design for reinforced concrete will prove of little effect in securing thoroughly good construction unless the general procedure to be followed in construction is anticipated and controlled by the engineer responsible for the design. In structural steelwork, such general procedure can be taken for granted, but not in reinforced concrete. The writer, therefore, would suggest the following additional clause as desirable in general specifications for reinforced concrete design:

"The location and shape of the joints at which reinforced concrete work can be suspended, shall be determined by the designing engineer and indicated clearly on the plans, and the concrete, for any one of the portions into which the construction is thus divided, shall be deposited in one continuous operation."

This, of course, assumes that the work will be inspected thoroughly and continuously to insure the effective carrying out of the designer's intention.

In regard to the design of timber structures, the author's specifications depart radically from common practice, as they require that "all live strains must first be increased to equivalent static strains by formula" (which formula for spans under 20 ft. requires an addition of more than 100%), and the specifications only increase by 50% the ordinary allowable stresses, which have been commonly used without considering impact. This departure can be shown very strikingly by investigating the strength of an ordinary railroad timber trestle, as commonly built for *E*-40 loading, with bents 12 ft. 6 in. between centers, and two Georgia pine stringers, 9 by 16 in., under each rail, a construction which has amply proved its efficiency. Without considering impact, the calculated stress in the outer fiber of these stringers, in bending, is about 1 400 lb. per sq. in., but with impact included, according to the author's formula, this stress is 2 730 lb. per sq. in., or more than 50% in excess of the 1 800 lb. allowed in the specifications. Notwithstanding this, the following note is inserted under the tabulated "Allowable Static Strains for Timber":

"In this table the values for allowable static strains for timber are 50% greater than those formerly used for miscellaneous loading, and 100% greater than would be allowed for a moving load suddenly applied."

The specifications should certainly be revised as regards the design of timber structures, which are usually of short span.

Mr. Conover.

CHARLES E. CONOVER, M. AM. SOC. C. E. (by letter).—In connection with these specifications, as used in the office of the Chief Engineer of the Public Service Commission, the preparation of tables which would dispense with a large amount of duplication in calculations has long been considered by the writer to be of great importance.

As opportunities have offered, he has prepared tables bearing on the subject of plate girders which he thought would be of particular advantage and importance. Mr. Conover.

The first of this series, Table 4, covers "Net Areas of Girder Flanges" with 14, 16, 18, and 20-in. cover-plates. These tables of net areas are based on the deductions of two rivet holes for each plate and two (and three) rivet holes for each angle.

The second and third series, Tables 5 and 6, entitled "Weights of Girders per Linear Foot" (main material only), and "Weights of Two Cover-Plates of Varying Lengths and Thicknesses," are of much value in estimating steelwork, and have proved themselves great time savers.

TABLE 4.—NET AREAS OF GIRDER FLANGES.

2 1-in. Holes Deducted from Plates.
2 1-in. Holes Deducted from Angles.

SIZE OF COVER-PLATES.	NET AREA OF PLATES, IN SQUARE INCHES.	SIZE OF FLANGE ANGLES.				
		6 × 6 × 5/8"	6 × 6 × 11/16"	6 × 6 × 3/4"	6 × 6 × 13/16"	6 × 6 × 7/8"
Net area of 2 angles.		11.72	12.80	13.88	14.93	15.98
14 × 3/8"	4.50	16.22	17.30	18.38	19.43	20.48
7/16	5.25	16.97	18.05	19.13	20.18	21.23
1/2	6.00	17.72	18.80	19.88	20.93	21.98
9/16	6.75	18.47	19.55	20.63	21.68	22.73
5/8	7.50	19.22	20.30	21.38	22.43	23.48
11/16	8.25	19.97	21.05	22.13	23.18	24.23
3/4	9.00	20.72	21.80	22.88	23.93	24.98
13/16	9.75	21.47	22.55	23.63	24.68	25.73
7/8	10.50	22.22	23.30	24.38	25.43	26.48
1	12.00	23.72	24.80	25.88	26.93	27.98
1 1/8	13.50	25.22	26.30	27.38	28.43	29.48
1 1/4	15.00	26.72	27.80	28.88	29.93	30.98
1 3/8	16.50	28.22	29.30	30.38	31.43	32.48
1 1/2	18.00	29.72	30.80	31.88	32.93	33.98
1 5/8	19.50	31.22	32.30	33.38	34.43	35.48
1 3/4	21.00	32.72	33.80	34.88	35.93	36.98
1 7/8	22.50	34.22	35.30	36.38	37.43	38.48
2	24.00	35.72	36.80	37.88	38.93	39.98
16 × 3/8"	5.25	16.97	18.05	19.13	20.18	21.23
1/2	7.00	18.72	19.80	20.88	21.93	22.98
3/4	10.50	22.22	23.30	24.38	25.43	26.48
7/8	12.25	23.97	25.05	26.13	27.18	28.23
1	14.00	25.72	26.80	27.88	28.93	29.98
1 1/8	15.75	27.47	28.55	29.63	30.68	31.73
1 1/4	17.50	29.22	30.30	31.38	32.43	33.48
1 3/8	19.25	30.97	32.05	33.13	34.18	35.23
1 1/2	21.00	32.32	33.80	34.88	35.93	36.98
1 3/4	24.50	36.22	37.30	38.38	39.43	40.48
2	28.00	39.72	40.80	41.88	42.93	43.98
2 1/4	31.50	43.22	44.30	45.38	46.43	47.48
2 1/2	35.00	46.72	47.80	48.88	49.93	50.98
2 3/4	38.50	50.22	51.30	52.38	53.43	54.48
3	42.00	53.72	54.80	55.88	56.93	57.98

Mr. Conover.

TABLE 4.—Net Areas of Girder Flanges.—(*Continued.*)
2 1-in. Holes Deducted from Plates.
3 1-in. Holes Deducted from Angles.

Size of Cover-Plates.	Net Area of Plates, in Square Inches.	Size of Flange Angles.				
		8 × 8 × 5/8"	8 × 8 × 11/16"	8 × 8 × 3/4"	8 × 8 × 13/16"	8 × 8 × 7/8"
Net area of 2 angles.		15.47	16.92	18.38	19.82	21.20
18 × 3/8"	6.00	21.47	22.92	24.38	25.82	27.20
7/16	7.00	22.47	23.92	25.38	26.82	28.20
1/2	8.00	23.47	24.92	26.38	27.82	29.20
9/16	9.00	24.47	25.92	27.38	28.82	30.20
5/8	10.00	25.47	26.92	28.38	29.82	31.20
11/16	11.00	26.47	27.92	29.38	30.82	32.20
3/4	12.00	27.47	28.92	30.38	31.82	33.20
13/16	13.00	28.47	29.92	31.38	32.82	34.20
7/8	14.00	29.47	30.92	32.38	33.82	35.20
1	16.00	31.47	32.92	34.38	35.82	37.20
1 1/8	18.00	33.47	34.92	36.38	37.82	39.20
1 1/4	20.00	35.47	36.92	38.38	39.82	41.20
1 3/8	22.00	37.47	38.92	40.38	41.82	43.20
1 1/2	24.00	39.47	40.92	42.38	43.82	45.20
20 × 5/8"	11.25	26.72	28.17	29.63	31.07	32.45
11/16	12.38	27.85	29.30	30.76	32.20	33.58
3/4	13.50	28.97	30.42	31.88	33.32	34.70
13/16	14.63	30.10	31.55	33.01	34.45	35.83
7/8	15.75	31.22	32.67	34.13	35.57	36.95
1	18.00	33.47	34.92	36.38	37.82	39.20
1 1/8	20.25	35.72	37.17	38.63	40.07	41.45
1 1/4	22.50	37.97	39.42	40.88	42.32	43.70
1 3/8	24.75	40.22	41.67	43.13	44.57	45.95
1 1/2	27.00	42.47	43.92	45.38	46.82	48.20
1 5/8	29.25	44.72	46.17	47.63	49.07	50.45
1 3/4	31.50	46.97	48.42	49.88	51.32	52.70
1 7/8	33.75	49.22	50.67	52.13	53.57	54.95
2	36.00	51.47	52.92	54.38	55.82	57.20
2 1/4	40.50	55.97	57.42	58.88	60.32	61.70
2 1/2	45.00	60.47	61.92	63.38	64.82	66.20
2 3/4	49.50	64.97	66.42	67.88	69.32	70.70
3	54.00	69.47	70.92	72.38	73.82	75.20
3 1/4	58.50	73.97	75.42	76.88	78.32	79.70
3 1/2	63.00	78.47	79.92	81.38	82.82	84.20

The fourth series, Table 7, entitled "Section Moduli of Girders," is one which the writer deems to be of great importance, and its preparation, long contemplated by him, was begun when the opportunity offered, about 3 years ago. The use of these tables has shortened very materially the time spent in designing subway structures; and, which is also of importance in their preparation, the moment-of-inertia method has been used, thus eliminating one objection to the ordinary method of plate-girder design, namely, that in designing plate girders by the usual approximate center to center of gravity method, the section obtained is from 10 to 15% overstrained in the extreme fiber. These tables are based on deducting one hole (1 in.

diameter of metal) from each angle. Deduction for additional holes may readily be made by finding the section modulus of the material taken out, referring this section modulus, of course, to the extreme fiber of the girder, and subtracting this value from that of the section modulus as given in the tables. Mr. Conover.

The section modulus for a girder with cover-plates may also be found readily by remembering that the section modulus of that part of the girder, excluding the cover-plates, bears the direct ratio to the value given in the table that the depth, from out to out, of girders without cover-plates bears to the depth, from out to out, of girders with cover-plates.

TABLE 5.—WEIGHTS OF GIRDERS PER LINEAR FOOT.
4 Angles and Web.

4 ANGLES.	WEB, 24 INCHES.									
Size.	Weight.	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″
		30.6	35.7	40.8	45.9	51.0	56.1	61.2	66.3	71.4
5 × 3 1/2 × 3/8″	41.6	72.2	77.3	82.4	87.5	92.6	97.7	102.8	107.9	113.0
7/16	48.0	78.6	83.7	88.8	93.9	99.0	104.1	109.2	114.3	119.4
1/2	54.4	85.0	90.1	95.2	100.3	105.4	110.5	115.6	120.7	125.8
9/16	60.8	91.4	96.5	101.6	106.7	111.8	116.9	122.0	127.1	132.2
5/8	67.2	97.8	102.9	108.0	113.1	118.2	123.3	128.4	133.5	138.6
11/16	73.2	103.8	108.9	114.0	119.1	124.2	129.3	134.4	139.5	144.6
3/4	79.2	109.8	114.9	120.0	125.1	130.2	135.3	140.4	145.5	150.6
13/16	85.2	115.8	120.9	126.0	131.1	136.2	141.3	146.4	151.5	156.6
7/8	90.8	121.4	126.5	131.6	136.7	141.8	146.9	152.0	157.1	162.2
6 × 4 × 3/8″	49.2	79.8	84.9	90.0	95.1	100.2	105.3	110.4	115.5	120.6
7/16	57.2	87.8	92.9	98.0	103.1	108.2	113.3	118.4	123.5	128.6
1/2	64.8	95.4	100.5	105.6	110.7	115.8	120.9	126.0	131.1	136.2
9/16	72.4	103.0	108.1	113.2	118.3	123.4	128.5	133.6	138.7	143.8
5/8	80.0	110.6	115.7	120.8	125.9	131.0	136.1	141.2	146.3	151.4
11/16	87.2	117.8	122.9	128.0	133.1	138.2	143.3	148.4	153.5	158.6
3/4	94.4	125.0	130.1	135.2	140.3	145.4	150.5	155.6	160.7	165.8
13/16	101.6	132.2	137.3	142.4	147.5	152.6	157.7	162.8	167.9	173.0
7/8	108.8	139.4	144.5	149.6	154.7	159.8	164.9	170.0	175.1	180.2
6 × 6 × 3/8″	59.6	90.2	95.3	100.4	105.5	110.6	115.7	120.8	125.9	131.0
7/16	68.8	99.4	104.5	109.6	114.7	119.8	124.9	130.0	135.1	140.2
1/2	78.4	109.0	114.1	119.2	124.3	129.4	134.5	139.6	144.7	149.8
9/16	87.6	118.2	123.3	128.4	133.5	138.6	143.7	148.8	153.9	159.0
5/8	96.8	127.4	132.5	137.6	142.7	147.8	152.9	158.0	163.1	168.2
11/16	106.0	136.6	141.7	146.8	151.9	157.0	162.1	167.2	172.3	177.4
3/4	114.8	145.4	150.5	155.6	160.7	165.8	170.9	176.0	181.1	186.2
13/16	124.0	154.6	159.7	164.8	169.9	175.0	180.1	185.2	190.3	195.4
7/8	132.4	163.0	168.1	173.2	178.3	183.4	188.5	193.6	198.7	203.8
8 × 8 × 1/2″	105.6	136.2	141.3	146.4	151.5	156.6	161.7	166.8	171.8	177.0
9/16	118.4	149.0	154.1	159.2	164.3	169.4	174.5	179.6	184.7	189.8
5/8	130.8	161.4	166.5	171.6	176.7	181.8	186.9	192.0	197.1	202.2
11/16	143.2	173.8	178.9	184.0	189.1	194.2	199.3	204.4	209.5	214.6
3/4	155.6	186.2	191.3	196.4	201.5	206.6	211.7	216.8	221.9	227.0
13/16	168.0	198.6	203.7	208.8	213.9	219.0	224.1	229.2	234.3	239.4
7/8	180.0	210.6	215.7	220.8	225.9	231.0	236.1	241.2	246.3	251.4

Mr. Conover.

TABLE 5.—WEIGHTS OF GIRDERS PER LINEAR FOOT.—(*Continued.*)
4 Angles and Web.

4 ANGLES.	WEB, 27 INCHES.									
Size.	Weight.	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″
		34.4	40.2	45.9	51.6	57.4	63.1	68.9	74.6	80.3
5 × 3 1/2 × 3/8″	41.6	76.0	81.8	87.5	93.2	99.0	104.7	110.5	116.2	121.9
7/16	48.0	82.4	88.2	93.9	99.6	105.4	111.1	116.9	122.6	128.3
1/2	54.4	88.8	94.6	100.3	106.0	111.8	117.5	123.3	129.0	134.7
9/16	60.8	95.2	101.0	106.7	112.4	118.2	123.9	129.7	135.4	141.1
5/8	67.2	101.6	107.4	113.1	118.8	124.6	130.3	136.1	141.8	147.5
11/16	73.2	107.6	113.4	119.1	124.8	130.6	136.3	142.1	147.8	153.5
3/4	79.2	113.6	119.4	125.1	130.8	136.6	142.3	148.1	153.8	159.5
13/16	85.2	119.6	125.4	131.1	136.8	142.6	148.3	154.1	159.8	165.5
7/8	90.8	125.2	131.0	136.7	142.4	148.2	153.9	159.7	165.4	171.1
6 × 4 × 3/8″	49.2	83.6	89.4	95.1	100.8	106.6	112.3	118.1	123.8	129.5
7/16	57.2	91.6	97.4	103.1	108.8	114.6	120.3	126.1	131.8	137.5
1/2	64.8	99.2	105.0	110.7	116.4	122.2	127.9	133.7	139.4	145.1
9/16″	72.4	106.8	112.6	118.3	124.0	129.8	135.5	141.3	147.0	152.7
5/8	80.0	114.4	120.2	125.9	131.6	137.4	143.1	148.9	154.6	160.3
11/16	87.2	121.6	127.4	133.1	138.8	144.6	150.3	156.1	161.8	167.5
3/4	94.4	128.8	134.6	140.3	146.0	151.8	157.5	163.3	169.0	174.7
13/16	101.6	136.0	141.8	147.5	153.2	159.0	164.9	170.5	176.2	181.9
7/8	108.8	143.2	149.0	154.7	160.4	166.2	171.9	177.7	183.4	189.1
6 × 6 × 3/8″	59.6	94.0	99.8	105.5	111.2	117.0	122.7	128.5	134.2	139.9
7/16	68.8	103.2	109.0	114.7	120.4	126.2	131.9	137.7	143.4	149.1
1/2	78.4	112.8	118.6	124.3	130.0	135.8	141.5	147.3	153.0	158.7
9/16	87.6	122.0	127.8	133.5	139.2	145.0	150.7	156.3	162.2	167.9
5/8	96.8	131.2	137.0	142.7	148.4	154.2	159.9	165.7	171.4	177.1
11/16	106.0	140.4	146.2	151.9	157.6	163.4	169.1	174.9	180.6	186.3
3/4	114.8	149.2	155.0	160.7	166.4	172.2	177.9	183.7	189.4	195.1
13/16	124.0	158.4	164.2	169.9	175.6	181.4	187.1	192.9	198.6	204.3
7/8	132.4	166.8	172.6	178.3	184.0	189.8	195.5	201.3	207.0	212.7
8 × 8 × 1/2″	105.6	140.0	145.8	151.5	157.2	163.0	168.7	174.5	180.2	185.9
9/16	118.4	152.8	158.6	164.3	170.0	175.8	181.5	187.3	193.0	198.7
5/8	130.8	165.2	171.0	176.7	182.4	188.2	193.9	199.7	205.4	211.1
11/16	143.2	177.6	183.4	189.1	194.8	200.6	206.3	212.1	217.8	223.5
3/4	155.6	190.0	195.8	201.5	207.2	213.0	218.7	224.5	230.2	235.9
13/16	168.0	202.4	208.2	213.9	219.6	225.4	231.1	236.9	242.6	248.3
7/8	180.0	214.4	220.2	225.9	231.6	237.4	243.1	248.9	254.6	260.3

4 ANGLES.	WEB, 30 INCHES.									
Size.	Weight.	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″
		38.3	44.6	51.0	57.4	63.8	70.1	76.5	82.9	89.2
6 × 6 × 3/8″	59.6	97.9	104.2	110.6	117.0	123.4	129.7	136.1	142.5	148.8
7/16	68.8	107.1	113.4	119.8	126.2	132.6	138.9	145.3	151.7	158.0
1/2	78.4	116.7	123.0	129.4	135.8	142.2	148.5	154.9	161.3	167.6
9/16	87.6	125.9	132.2	138.6	145.0	151.4	157.7	164.1	170.5	176.8
5/8	96.8	135.1	141.4	147.8	154.2	160.6	166.9	173.3	179.7	186.0
11/16	106.0	144.3	150.6	157.0	163.4	169.8	176.1	182.5	188.9	195.2
3/4	114.8	153.1	159.4	165.8	172.2	178.6	184.9	191.3	197.7	204.0
13/16	124.0	162.3	168.6	175.0	181.4	187.8	194.1	200.5	206.9	213.2
7/8	132.4	170.7	177.0	183.4	189.8	196.2	202.5	208.9	215.3	221.6
8 × 8 × 1/2″	105.6	143.9	150.2	156.6	163.0	169.4	175.7	182.1	188.5	194.8
9/16	118.4	156.7	163.0	169.4	175.8	182.2	188.5	194.9	201.3	207.6
5/8	130.8	169.1	175.4	181.8	188.2	194.6	200.9	207.3	213.7	220.0
11/16	143.2	181.5	187.8	194.2	200.6	207.0	213.3	219.7	226.1	232.4
3/4	155.6	193.9	200.2	206.6	213.0	219.4	225.7	232.1	238.5	244.8
13/16	168.0	206.3	212.6	219.0	225.4	231.8	238.1	244.5	250.9	257.2
7/8	180.0	218.3	224.6	231.0	237.4	243.8	250.1	256.5	262.9	269.2

Mr. Conover.

TABLE 5.—WEIGHTS OF GIRDERS PER LINEAR FOOT.—(*Continued.*)
4 Angles and Web.

4 Angles.		Web, 33 Inches.								
Size.	Weight.	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″
		42.1	49.1	56.1	63.1	70.1	77.1	84.1	91.1	98.2
6 × 6 × 3/8″	59.6	101.7	108.7	115.7	122.7	129.7	136.7	143.7	150.7	157.7
7/16	68.8	110.9	117.9	124.9	131.9	138.9	145.9	152.9	159.9	167.0
1/2	78.4	120.5	127.5	134.5	141.5	148.5	155.5	162.5	169.5	176.6
9/16	87.6	129.7	136.7	143.7	150.7	157.7	164.7	171.7	178.7	185.8
5/8	96.8	138.9	145.9	152.9	159.9	166.9	173.9	180.9	187.9	195.0
11/16	106.0	148.1	155.1	162.1	169.1	176.1	183.1	190.1	197.1	204.2
3/4	114.8	156.9	163.9	170.9	177.9	184.9	191.9	198.9	205.9	213.0
13/16	124.0	166.1	173.1	180.1	187.1	194.1	201.1	208.1	215.1	222.2
7/8	132.4	174.5	181.5	188.5	195.5	202.5	209.5	216.5	223.5	230.5
8 × 8 × 1/2″	105.6	147.7	154.7	161.7	168.7	175.7	182.7	189.7	196.7	203.7
9/16	118.4	160.5	167.5	174.5	181.5	188.5	195.5	202.5	209.5	216.5
5/8	130.8	172.9	179.9	186.9	193.9	200.9	207.9	214.9	221.9	229.0
11/16	143.2	185.3	192.3	199.3	206.3	213.3	220.3	227.3	234.3	241.4
3/4	155.6	197.7	204.7	211.7	218.7	225.7	232.1	239.7	246.7	253.8
13/16	168.0	210.1	217.1	224.1	231.1	238.1	245.1	252.1	259.1	266.2
7/8	180.0	222.1	229.1	236.1	243.1	250.1	257.1	264.1	271.1	278.2

4 Angles.		Web, 36 Inches.								
Size.	Weight.	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″
		45.9	53.6	61.2	68.9	76.5	84.2	91.8	99.4	107.1
6 × 6 × 3/8″	59.6	105.5	113.2	120.8	128.5	136.1	143.8	151.4	159.0	166.7
7/16	68.8	114.7	122.3	130.0	137.7	145.3	153.0	160.6	168.2	175.9
1/2	78.4	124.3	132.0	139.6	147.3	154.9	162.6	170.2	177.8	185.5
9/16	87.6	133.5	141.2	148.8	156.5	164.1	171.8	179.4	187.0	194.7
5/8	96.8	142.7	150.4	158.0	165.7	173.3	181.0	188.6	196.2	203.9
11/16	106.0	151.9	159.6	167.2	174.9	182.5	190.2	197.8	205.4	213.1
3/4	114.8	160.7	168.4	176.0	183.7	191.3	199.0	206.6	214.2	221.9
13/16	124.0	169.9	177.6	185.2	192.9	200.5	208.2	215.8	223.4	231.1
7/8	132.4	178.3	186.0	193.6	201.3	208.9	216.6	224.2	231.8	239.5
8 × 8 × 1/2″	105.6	151.5	159.2	166.8	174.5	182.1	189.8	197.4	205.0	212.7
9/16	118.4	164.3	172.0	179.6	187.2	194.9	202.6	210.2	217.8	225.5
5/8	130.8	176.7	184.4	192.0	199.7	207.3	215.0	222.6	230.2	237.9
11/16	143.2	189.1	196.8	204.4	212.1	219.7	227.4	235.0	242.6	250.3
3/4	155.6	201.5	209.2	216.8	224.5	232.1	239.8	247.4	255.0	262.7
13/16	168.0	213.9	221.6	229.2	236.9	244.5	252.2	259.8	267.4	275.1
7/8	180.0	225.9	233.6	241.2	248.9	256.5	264.2	271.8	279.4	287.1

Mr. Conover.

TABLE 5.—Weights of Girders per Linear Foot.—(*Continued.*)
4 Angles and Web.

4 Angles.		Web, 39 Inches.								
Size.	Weight.	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″
		49.8	58.1	66.3	74.6	82.9	91.6	99.5	107.7	116.1
6 × 6 × 3/8″	59.6	109.4	117.7	125.9	134.2	142.5	150.8	159.1	167.3	175.7
7/16	68.8	118.6	126.9	135.1	143.4	151.7	160.0	168.3	176.5	184.9
1/2	78.4	128.2	136.5	144.7	153.0	161.3	169.6	177.9	186.1	194.5
9/16	87.6	137.4	145.7	153.9	162.2	170.5	178.8	187.1	195.3	203.7
5/8	96.8	146.6	154.9	163.1	171.4	179.7	188.0	196.3	204.5	212.9
11/16	106.0	155.8	164.1	172.3	180.6	188.9	197.2	205.5	213.7	222.1
3/4	114.8	164.6	172.9	181.1	189.4	197.7	206.0	214.3	222.5	230.9
13/16	124.0	173.8	182.1	190.3	198.6	206.9	215.2	223.5	231.7	240.1
7/8	132.4	182.2	190.5	198.7	207.0	215.3	223.6	231.9	240.1	248.5
8 × 8 × 1/2″	105.6	155.4	163.7	171.9	180.2	188.5	196.8	205.1	213.3	221.7
9/16	118.4	168.2	176.5	184.7	193.0	201.3	209.6	217.9	226.1	234.5
5/8	130.8	180.6	188.9	197.1	205.4	213.7	222.0	230.3	238.5	246.9
11/16	143.2	193.0	201.3	209.5	217.8	226.1	234.4	242.7	250.9	259.3
3/4	155.6	205.4	213.7	221.9	230.2	238.5	246.8	255.1	263.3	271.7
13/16	168.0	217.8	226.1	234.3	242.6	250.9	259.2	267.5	275.7	284.1
7/8	180.0	229.8	238.1	246.3	254.6	262.9	271.2	279.5	287.7	296.1
4 Angles.		**Web, 42 Inches.**								
Size.	Weight.	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″
		53.6	62.5	71.4	80.3	89.3	98.2	107.1	116.1	125.0
6 × 6 × 3/8″	59.6	113.2	122.1	131.0	139.9	148.9	157.8	166.7	175.7	184.6
7/16	68.8	122.4	131.3	140.2	149.1	158.1	167.0	175.9	184.9	193.8
1/2	78.4	132.0	140.9	149.8	158.7	167.7	176.6	185.5	194.5	203.4
9/16	87.6	141.2	150.1	159.0	167.9	176.9	185.8	194.7	203.7	212.6
5/8	96.8	150.4	159.3	168.2	177.1	186.1	195.0	203.9	212.9	221.8
11/16	106.0	159.6	168.5	177.4	186.3	195.3	204.2	213.1	222.1	231.0
3/4	114.8	168.4	177.3	186.2	195.1	204.1	213.0	221.9	230.9	239.8
13/16	124.0	177.6	186.5	195.4	204.3	213.3	222.2	231.1	240.1	249.0
7/8	132.4	186.0	194.9	203.8	212.7	221.7	230.6	239.5	248.5	257.4
8 × 8 × 1/2″	105.6	159.2	168.1	177.0	185.9	194.9	203.8	212.7	221.7	230.6
9/16	118.4	172.0	180.9	189.8	198.7	207.7	216.6	225.5	234.5	243.4
5/8	130.8	184.4	193.3	202.2	211.1	220.1	229.0	237.9	216.9	255.8
11/16	143.2	196.8	205.7	214.6	223.5	232.5	241.4	250.3	259.3	268.2
3/4	155.6	209.2	218.1	227.0	235.9	244.9	253.8	262.7	271.7	280.6
13/16	168.0	221.6	230.5	239.4	248.3	257.3	266.2	275.1	284.1	293.0
7/8	180.0	233.6	242.5	251.4	260.3	269.3	278.2	287.1	296.1	305.0

Mr. Conover.

TABLE 5.—Weights of Girders per Linear Foot.—(*Continued.*)
4 Angles and Web.

4 Angles. Size.	Web, 45 Inches. Weight.	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″
		57.4	67.0	76.5	86.1	95.7	105.2	114.8	124.3	133.9
6 × 6 × 3/8″	59.6	117.0	126.6	136.1	145.7	155.3	164.8	174.4	183.9	193.5
7/16	68.8	126.2	135.8	145.3	154.9	164.5	174.0	183.6	193.1	202.7
1/2	78.4	135.8	145.4	154.9	164.5	174.1	183.6	193.2	202.7	212.3
9/16	87.6	145.0	154.6	164.1	173.7	183.3	192.8	202.4	211.9	221.5
5/8	96.8	154.2	163.8	173.3	182.9	192.5	202.0	211.6	221.1	230.7
11/16	106.0	163.4	173.0	182.5	192.1	201.7	211.2	220.8	230.3	239.9
3/4	114.8	172.2	181.8	191.3	200.9	210.5	220.0	229.6	239.1	248.7
13/16	124.0	181.4	191.0	200.5	210.1	219.7	229.2	238.8	248.3	257.9
7/8	132.4	189.8	199.4	208.9	218.5	228.1	237.6	247.2	256.7	266.3
8 × 8 × 1/2″	105.6	163.0	172.6	182.1	191.7	201.3	210.8	220.4	229.9	239.5
9/16	118.4	175.8	185.4	194.9	204.5	214.1	223.6	233.2	242.7	252.3
5/8	130.8	188.2	197.8	207.3	216.9	226.5	236.0	245.6	255.1	264.7
11/16	143.2	200.6	210.2	219.7	229.3	238.9	248.4	258.0	267.5	277.1
3/4	155.6	213.0	222.6	232.1	241.7	251.3	260.8	270.4	279.9	289.5
13/16	168.0	225.4	235.0	244.5	254.1	263.7	273.2	282.8	292.3	301.9
7/8	180.0	237.4	247.0	256.5	266.1	275.7	285.2	294.8	304.3	313.9

4 Angles. Size.	Web, 48 Inches. Weight.	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″
		61.2	71.5	81.6	91.8	102.0	112.2	122.4	132.6	142.8
6 × 6 × 3/8″	59.6	120.8	131.0	141.2	151.4	161.6	171.8	182.0	192.2	202.4
7/16	68.8	130.0	140.2	150.4	160.6	170.8	181.0	191.2	201.4	211.6
1/2	78.4	139.6	149.8	160.0	170.2	180.4	190.6	200.8	211.0	221.2
9/16	87.6	148.8	159.0	169.2	179.4	189.6	199.8	210.0	220.2	230.4
5/8	96.8	158.0	168.2	178.4	188.6	198.8	209.0	219.2	229.4	239.6
11/16	106.0	167.2	177.4	187.6	197.8	208.0	218.2	228.4	238.6	248.8
3/4	114.8	176.0	186.2	196.4	206.6	216.8	227.0	237.2	247.4	257.6
13/16	124.0	185.2	195.4	205.6	215.8	226.0	236.2	246.4	256.6	266.8
7/8	132.4	193.6	203.8	214.0	224.2	234.4	244.6	254.8	265.0	275.2
8 × 8 × 1/2″	105.6	166.8	177.0	187.2	197.4	207.6	217.8	228.0	238.2	248.4
9/16	118.4	179.6	189.8	200.0	210.2	220.4	230.6	240.8	251.0	261.2
5/8	130.8	192.0	202.2	212.4	222.6	232.8	243.0	253.2	263.4	273.6
11/16	143.2	204.4	214.6	224.8	235.0	245.2	255.4	265.6	275.8	286.0
3/4	155.6	216.8	227.0	237.2	247.4	257.6	267.8	278.0	288.2	298.4
13/16	168.0	229.2	239.4	249.6	259.8	270.0	280.2	290.4	300.6	310.8
7/8	180.0	241.2	251.4	261.6	271.8	282.0	292.2	302.4	312.6	322.8

Mr. Conover.

TABLE 5.—Weights of Girders per Linear Foot.—(*Continued.*)

4 Angles and Web.

4 Angles.	Web, 51 Inches.									
Size.	Weight.	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″
		65.1	75.9	86.7	97.6	108.4	119.3	130.1	140.9	151.7
6 × 6 × 3/8″	59.6	124.7	135.5	146.3	157.2	168.0	178.9	189.7	200.5	211.3
7/16	68.8	133.9	144.7	155.5	166.4	177.2	188.1	198.9	209.7	220.5
1/2	78.4	143.5	154.3	165.1	176.0	186.8	197.7	208.5	219.3	230.1
9/16	87.6	152.7	163.5	174.3	185.2	196.0	206.9	217.7	228.5	239.3
5/8	96.8	161.9	172.7	183.5	194.4	205.2	216.1	226.9	237.7	248.5
11/16	106.0	171.1	181.9	192.7	203.6	214.4	225.3	236.1	246.9	257.7
3/4	114.8	179.9	190.7	201.5	212.4	223.2	234.1	244.9	255.7	266.5
13/16	124.0	189.1	199.9	210.7	221.6	232.4	243.3	254.1	264.9	275.7
7/8	132.4	197.5	208.3	219.1	230.0	240.8	251.7	262.5	273.3	284.1
8 × 8 × 1/2″	105.6	170.7	181.5	192.3	203.2	214.0	224.9	235.7	246.5	257.3
9/16	118.4	183.5	194.3	205.1	216.0	226.8	237.7	248.5	259.3	270.1
5/8″	130.8	195.9	206.7	217.5	228.4	239.2	250.1	260.9	271.7	282.5
11/16	143.2	208.3	219.1	229.9	240.8	251.6	262.5	273.3	284.1	294.9
3/4	155.6	220.7	231.5	242.3	253.2	264.0	274.9	285.7	296.5	307.3
13/16	168.0	233.1	243.9	254.7	265.6	276.4	287.3	298.1	308.9	319.7
7/8	180.0	245.1	255.9	266.7	277.6	288.4	299.3	310.1	320.9	331.7

4 Angles.	Web, 54 Inches.									
Size.	Weight.	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″
		68.9	80.3	91.8	103.3	114.7	126.2	137.8	149.2	160.7
6 × 6 × 3/8″	59.6	128.5	139.9	151.4	162.9	174.3	185.8	197.4	208.8	220.3
7/16	68.8	137.7	149.1	160.6	172.1	183.5	195.0	206.6	218.0	229.5
1/2	78.4	147.3	158.7	170.2	181.7	193.1	204.6	216.2	227.6	239.1
9/16	87.6	156.5	167.9	179.4	190.9	202.3	213.8	225.4	236.8	248.3
5/8	96.8	165.7	177.1	188.6	200.1	211.5	223.0	234.6	246.0	257.5
11/16	106.0	174.9	186.3	197.8	209.3	220.7	232.2	243.8	255.2	266.7
3/4	114.8	183.7	195.1	206.6	218.1	229.5	241.0	252.6	264.0	275.5
13/16	124.0	192.9	204.3	215.8	227.3	238.7	250.2	261.8	273.2	284.7
7/8	132.4	201.3	212.7	224.2	235.7	247.1	258.6	270.2	281.6	293.1
8 × 8 × 1/2″	105.6	174.5	185.9	197.4	208.9	220.3	231.8	243.4	254.8	266.3
9/16	118.4	187.3	198.7	210.2	221.7	233.1	244.6	256.2	267.6	279.1
5/8	130.8	199.7	211.1	222.6	234.1	245.5	257.0	268.6	280.0	291.5
11/16	143.2	212.1	223.5	235.0	246.5	257.9	269.4	281.0	292.4	303.9
3/4	155.6	224.5	235.9	247.4	258.9	270.3	281.8	293.4	304.8	316.3
13/16	168.0	236.9	248.3	259.8	271.3	282.7	294.2	305.8	317.2	328.7
7/8	180.0	248.9	260.3	271.8	283.3	294.7	306.2	317.8	329.2	340.7

TABLE 5.—WEIGHTS OF GIRDERS PER LINEAR FOOT.—(*Continued.*) Mr. Conover.

4 Angles and Web.

4 ANGLES.	WEB, 57 INCHES.									
Size.	Weight.	3/8"	7/16"	1/2"	9/16"	5/8"	11/16"	3/4"	13/16"	7/8"
		72.6	84.8	96.9	109.0	121.1	133.2	145.4	157.5	169.6
6 × 6 × 3/8"	59.6	132.2	144.4	156.5	168.6	180.7	192.8	205.0	217.1	229.2
7/16	68.8	141.4	153.6	165.7	177.8	189.9	202.0	214.2	226.3	238.4
1/2	78.4	151.0	163.2	175.3	187.4	199.5	211.6	223.8	235.9	248.0
9/16	87.6	160.2	172.4	184.5	196.6	208.7	220.8	233.0	245.1	257.2
5/8	96.8	169.4	181.6	193.7	205.8	217.9	230.0	242.1	254.2	266.3
11/16	106.0	178.6	190.7	202.8	214.9	227.0	239.1	251.2	263.3	275.4
3/4	114.8	187.4	199.6	211.7	223.8	235.9	248.0	260.2	272.3	284.4
13/16	124.0	196.6	208.7	220.8	232.9	245.0	257.1	269.2	281.3	293.4
7/8	132.4	205.0	217.1	229.1	241.2	253.3	265.4	277.5	289.6	301.7
8 × 8 × 1/2"	105.6	178.2	190.4	202.5	214.6	226.7	238.8	250.9	263.0	275.1
9/16	118.4	191.0	203.1	215.2	227.3	239.4	251.5	263.6	275.7	287.8
5/8	130.8	203.4	215.5	227.6	239.7	251.8	263.9	276.1	288.2	300.3
11/16	143.2	215.8	227.9	240.0	252.1	264.2	276.3	288.4	300.5	312.6
3/4	155.6	228.2	240.3	252.4	264.5	276.6	288.7	300.8	312.9	325.0
13/16	168.0	240.6	252.7	264.8	276.9	289.0	301.1	313.2	325.3	337.4
7/8	180.0	252.6	264.7	276.8	289.1	301.0	313.1	325.2	337.3	349.4

4 ANGLES.	WEB, 60 INCHES.									
Size.	Weight.	3/8"	7/16"	1/2"	9/16"	5/8"	11/16"	3/4"	13/16"	7/8"
		76.6	89.3	102.0	114.8	127.5	140.3	153.1	165.7	178.5
6 × 6 × 3/8"	59.6	136.2	148.9	161.6	174.4	187.1	199.9	212.7	225.3	238.1
7/16	68.8	145.4	158.1	170.8	183.6	196.3	209.1	221.9	234.5	247.3
1/2	78.4	155.0	167.7	180.4	193.2	205.9	218.7	231.5	244.1	256.9
9/16	87.6	164.2	176.9	189.6	202.4	215.1	227.9	240.7	253.3	266.1
5/8	96.8	173.4	186.1	198.8	211.6	224.3	237.1	249.9	262.5	275.3
11/16	106.0	182.6	195.3	208.0	220.8	233.5	246.3	259.1	271.7	284.5
3/4	114.8	191.4	204.1	216.8	229.6	242.3	255.1	267.9	280.5	293.3
13/16	124.0	200.6	213.3	226.0	238.8	251.5	264.3	277.1	289.7	302.5
7/8	132.4	209.0	221.7	234.4	247.2	259.9	272.7	285.5	298.1	310.9
8 × 8 × 1/2"	105.6	182.2	194.9	207.6	220.4	233.1	245.9	258.7	271.3	284.1
9/16	118.4	195.0	207.7	220.4	233.2	245.9	258.7	271.5	284.1	296.9
5/8	130.8	207.4	220.1	232.8	245.6	258.3	271.1	283.9	296.5	309.3
11/16	143.2	219.8	232.5	245.2	258.0	270.7	283.5	296.3	308.9	321.7
3/4	155.6	232.2	244.9	257.6	270.4	283.1	295.9	308.7	321.3	334.1
13/16	168.0	244.6	257.3	270.0	282.8	295.5	308.3	321.0	333.7	346.5
7/8	180.0	256.6	269.3	282.0	294.8	307.5	320.3	333.1	345.7	358.5

Mr. Conover.

TABLE 6.—WEIGHTS OF TWO COVER-PLATES OF VARYING LENGTHS AND THICKNESSES.

14-Inch Cover-Plates.

L	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″
8.	286	333	381	428	476	524	571	619	666
8.5	304	354	405	455	506	556	607	658	708
9	321	375	428	482	535	589	643	696	750
9.5	340	396	452	509	565	622	678	786	792
10	357	416	476	536	595	654	714	773	832
10.5	375	437	500	562	625	687	750	812	874
11	393	458	524	589	654	720	786	851	916
11.5	411	479	547	616	684	753	821	890	958
12	429	500	571	643	714	785	857	939	1 000
12.5	447	521	595	670	744	818	893	968	1 042
13	464	541	619	696	773	851	928	1 005	1 082
13.5	482	562	643	723	803	883	964	1 044	1 124
14	500	583	666	750	833	916	1 000	1 083	1 166
14.5	518	604	690	777	862	949	1 036	1 122	1 208
15	536	625	714	803	892	982	1 071	1 161	1 250
15.5	554	645	738	830	922	1 014	1 107	1 199	1 290
16	572	666	762	857	952	1 047	1 143	1 238	1 332
16.5	590	687	785	884	981	1 080	1 178	1 277	1 374
17	607	708	809	911	1 011	1 112	1 214	1 315	1 416
17.5	625	729	833	937	1 040	1 145	1 250	1 354	1 458
18	643	750	857	964	1 071	1 178	1 286	1 393	1 500
18.5	661	770	881	991	1 100	1 210	1 321	1 431	1 540
19	679	791	904	1 018	1 131	1 243	1 357	1 470	1 582
19.5	697	812	928	1 044	1 160	1 276	1 393	1 508	1 622
20	714	832	952	1 071	1 190	1 309	1 428	1 546	1 664
20.5	732	854	976	1 098	1 219	1 342	1 464	1 586	1 708
21	750	874	1 000	1 125	1 250	1 374	1 500	1 624	1 748
21.5	768	895	1 023	1 151	1 279	1 407	1 536	1 663	1 790
22	786	916	1 047	1 178	1 309	1 440	1 571	1 702	1 832
22.	804	937	1 071	1 205	1 338	1 472	1 607	1 741	1 874
23	822	958	1 095	1 232	1 369	1 505	1 643	1 780	1 916
23.5	839	979	1 119	1 259	1 398	1 538	1 678	1 818	1 958
24	857	999	1 142	1 285	1 428	1 571	1 714	1 856	1 998
24.5	875	1 020	1 166	1 312	1 457	1 605	1 750	1 895	2 040
25	893	1 041	1 190	1 340	1 488	1 636	1 786	1 934	2 082
25.5	911	1 062	1 214	1 366	1 517	1 669	1 821	1 973	2 124
26	929	1 083	1 238	1 393	1 547	1 701	1 857	2 012	2 166
26.5	947	1 103	1 261	1 419	1 576	1 734	1 893	2 050	2 206
27	964	1 124	1 285	1 446	1 606	1 767	1 928	2 088	2 248
27.5	982	1 145	1 309	1 472	1 636	1 800	1 964	2 127	2 290
28	1 000	1 166	1 333	1 500	1 665	1 832	2 000	2 166	2 332
28.5	1 018	1 187	1 357	1 526	1 695	1 865	2 035	2 205	2 374
29	1 036	1 208	1 380	1 553	1 725	1 898	2 071	2 244	2 416
29.5	1 054	1 228	1 404	1 580	1 755	1 930	2 107	2 282	2 456
30	1 072	1 249	1 428	1 607	1 784	1 963	2 143	2 321	2 498
30.5	1 089	1 270	1 452	1 634	1 814	1 996	2 178	2 359	2 540
31	1 107	1 291	1 476	1 660	1 844	2 029	2 214	2 398	2 582
31.5	1 125	1 312	1 499	1 687	1 874	2 061	2 250	2 437	2 624
32	1 143	1 332	1 523	1 714	1 904	2 094	2 285	2 475	2 664
32.5	1 161	1 353	1 547	1 741	1 933	2 127	2 321	2 514	2 706
33	1 179	1 374	1 571	1 767	1 963	2 169	2 357	2 553	2 748
33.5	1 197	1 395	1 594	1 794	1 993	2 192	2 393	2 592	2 790
34	1 214	1 416	1 618	1 821	2 023	2 225	2 428	2 630	2 832
34.5	1 232	1 437	1 642	1 848	2 052	2 258	2 464	2 669	2 874
35	1 250	1 457	1 666	1 875	2 082	2 290	2 500	2 707	2 914
35.5	1 268	1 478	1 690	1 901	2 112	2 328	2 535	2 746	2 956
36	1 286	1 499	1 714	1 928	2 141	2 356	2 571	2 785	2 998
36.5	1 304	1 520	1 737	1 955	2 171	2 389	2 607	2 824	3 040
37	1 322	1 541	1 761	1 982	2 201	2 421	2 643	2 863	3 082
37.5	1 340	1 562	1 785	2 009	2 231	2 454	2 678	2 902	3 124
38	1 357	1 582	1 809	2 035	2 260	2 487	2 714	2 939	3 164
38.5	1 375	1 603	1 833	2 062	2 290	2 519	2 750	2 978	3 206
39	1 393	1 624	1 856	2 089	2 320	2 552	2 785	3 017	3 248
39.5	1 411	1 645	1 880	2 116	2 349	2 585	2 821	3 056	3 290
40	1 429	1 666	1 904	2 142	2 379	2 618	2 857	3 095	3 332
40.5	1 446	1 686	1 928	2 169	2 409	2 651	2 892	3 132	3 372
41	1 464	1 708	1 952	2 196	2 438	2 684	2 928	3 170	3 416
41.5	1 482	1 728	1 976	2 223	2 469	2 716	2 964	3 210	3 456
42	1 500	1 748	2 000	2 250	2 500	2 748	3 000	3 248	3 496
42.5	1 518	1 769	2 023	2 276	2 529	2 781	3 036	3 287	3 538
43	1 536	1 790	2 046	2 302	2 558	2 814	3 072	3 326	3 580
43.5	1 554	1 811	2 070	2 329	2 588	2 847	3 107	3 365	3 622

TABLE 6.—WEIGHTS OF TWO COVER-PLATES OF VARYING LENGTHS AND THICKNESSES.—(*Continued.*) Mr. Conover.

14-Inch Cover-Plates.

L	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″
44	1 572	1 832	2 094	2 356	2 618	2 880	3 142	3 404	3 664
44.5	1 590	1 853	2 118	2 383	2 647	2 912	3 178	3 443	3 706
45	1 608	1 874	2 142	2 410	2 676	2 944	3 214	3 482	3 748
45.5	1 626	1 895	2 166	2 437	2 707	2 977	3 250	3 521	3 790
46	1 644	1 916	2 190	2 464	2 738	3 010	3 286	3 560	3 832
46.5	1 661	1 937	2 214	2 491	2 767	3 043	3 321	3 598	3 874
47	1 678	1 958	2 238	2 518	2 796	3 076	3 356	3 636	3 916
47.5	1 696	1 978	2 261	2 544	2 826	3 109	3 392	3 674	3 956
48	1 714	1 998	2 284	2 570	2 856	3 142	3 428	3 712	3 996
48.5	1 732	2 019	2 308	2 597	2 885	3 176	3 464	3 751	4 038
49	1 750	2 040	2 332	2 624	2 914	3 210	3 500	3 790	4 080
49.5	1 768	2 061	2 356	2 652	2 945	3 241	3 536	3 829	4 122
50	1 786	2 082	2 380	2 680	2 976	3 272	3 572	3 868	4 164
50.5	1 804	2 103	2 404	2 706	3 005	3 305	3 607	3 907	4 206
51	1 822	2 124	2 428	2 732	3 034	3 338	3 642	3 946	4 248
51.5	1 840	2 145	2 452	2 759	3 064	3 370	3 678	3 985	4 290
52	1 858	2 166	2 476	2 786	3 094	3 402	3 714	4 024	4 332
52.5	1 876	2 186	2 499	2 812	3 123	3 435	3 750	4 062	4 372
53	1 894	2 206	2 522	2 838	3 152	3 468	3 786	4 100	4 412
53.5	1 911	2 227	2 546	2 865	3 182	3 501	3 821	4 138	4 454
54	1 928	2 248	2 570	2 892	3 212	3 534	3 856	4 176	4 496
54.5	1 946	2 269	2 594	2 918	3 242	3 567	3 892	4 215	4 538
55	1 964	2 290	2 618	2 944	3 272	3 600	3 928	4 254	4 580
55.5	1 982	2 311	2 642	2 972	3 301	3 632	3 964	4 293	4 622
56	2 000	2 332	2 666	3 000	3 330	3 664	4 000	4 332	4 664
56.5	2 018	2 353	2 690	3 025	3 360	3 697	4 035	4 371	4 706
57	2 036	2 374	2 714	3 052	3 390	3 730	4 070	4 410	4 748
57.5	2 054	2 395	2 737	3 079	3 420	3 763	4 106	4 449	4 790
58	2 072	2 416	2 760	3 106	3 450	3 796	4 142	4 488	4 832
58.5	2 090	2 436	2 784	3 133	3 480	3 828	4 178	4 526	4 872
59	2 108	2 456	2 808	3 160	3 510	3 860	4 214	4 564	4 912
59.5	2 126	2 477	2 832	3 187	3 539	3 893	4 250	4 603	4 954
60	2 144	2 498	2 856	3 214	3 568	3 926	4 286	4 642	4 996

18-Inch Cover-Plates.

L	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″
8	367	429	490	551	612	673	735	796	858
8.5	390	455	520	585	650	715	781	845	910
9	415	482	551	620	689	757	827	897	964
9.5	436	509	581	654	727	800	872	945	1 018
10	459	536	612	689	765	842	918	995	1 072
10.5	482	563	643	723	803	884	964	1 045	1 126
11	505	589	673	757	842	926	1 010	1 094	1 178
11.5	528	616	704	792	880	968	1 056	1 144	1 232
12	551	643	734	827	918	1 010	1 102	1 194	1 286
12.5	574	670	765	861	956	1 052	1 148	1 244	1 340
13	597	696	796	895	995	1 094	1 194	1 293	1 392
13.5	620	723	826	930	1 033	1 136	1 240	1 343	1 446
14	648	750	857	964	1 071	1 178	1 286	1 393	1 500
14.5	666	777	887	999	1 109	1 220	1 332	1 443	1 554
15	689	804	918	1 033	1 148	1 262	1 378	1 493	1 608
15.5	712	830	949	1 068	1 186	1 304	1 424	1 542	1 660
16	735	857	979	1 102	1 224	1 347	1 469	1 592	1 714
16.5	758	884	1 010	1 137	1 262	1 389	1 515	1 642	1 768
17	781	911	1 040	1 171	1 301	1 431	1 561	1 692	1 822
17.5	804	938	1 071	1 205	1 339	1 473	1 607	1 742	1 876
18	827	964	1 101	1 239	1 377	1 515	1 653	1 791	1 928
18.5	850	991	1 132	1 274	1 415	1 557	1 699	1 841	1 982
19	872	1 018	1 162	1 309	1 454	1 600	1 745	1 890	2 036
19.5	895	1 045	1 193	1 343	1 492	1 641	1 791	1 940	2 090
20	918	1 072	1 224	1 378	1 530	1 683	1 837	1 990	2 144
20.5	941	1 098	1 255	1 412	1 568	1 725	1 883	2 039	2 196
21	964	1 125	1 285	1 446	1 607	1 767	1 929	2 089	2 250
21.5	987	1 152	1 316	1 481	1 645	1 809	1 975	2 139	2 304
22	1 010	1 179	1 346	1 515	1 683	1 852	2 020	2 189	2 358
22.5	1 033	1 206	1 377	1 550	1 721	1 894	2 066	2 239	2 412
23	1 056	1 232	1 408	1 584	1 760	1 936	2 112	2 288	2 464
23.5	1 079	1 259	1 438	1 618	1 798	1 978	2 158	2 338	2 518
24	1 102	1 286	1 469	1 653	1 837	2 020	2 204	2 388	2 572

Mr. Conover.

TABLE 6.—WEIGHTS OF TWO COVER-PLATES OF VARYING LENGTHS AND THICKNESSES.—(*Continued.*)

18-Inch Cover-Plates.

L	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″
24.5	1 125	1 313	1 499	1 688	1 874	2 062	2 250	2 438	2 626
25	1 148	1 340	1 530	1 722	1 913	2 104	2 296	2 488	2 680
25.5	1 171	1 366	1 561	1 756	1 951	2 146	2 342	2 537	2 732
26	1 194	1 393	1 591	1 791	1 989	2 188	2 388	2 587	2 786
26.5	1 217	1 420	1 622	1 825	2 027	2 230	2 434	2 637	2 840
27	1 240	1 447	1 652	1 860	2 066	2 272	2 480	2 687	2 894
27.5	1 263	1 473	1 683	1 894	2 104	2 314	2 526	2 736	2 946
28	1 286	1 500	1 714	1 929	2 142	2 356	2 572	2 786	3 000
28.5	1 309	1 527	1 744	1 963	2 180	2 399	2 617	2 836	3 054
29	1 332	1 554	1 775	1 998	2 219	2 441	2 663	2 886	3 108
29.5	1 355	1 581	1 805	2 032	2 257	2 483	2 709	2 936	3 162
30	1 378	1 607	1 836	2 066	2 295	2 525	2 755	2 985	3 214
30.5	1 401	1 634	1 867	2 100	2 333	2 567	2 801	3 035	3 268
31	1 424	1 661	1 897	2 135	2 372	2 609	2 847	3 085	3 322
31.5	1 446	1 688	1 928	2 169	2 410	2 651	2 893	3 134	3 376
32	1 469	1 715	1 958	2 204	2 448	2 693	2 939	3 184	3 430
32.5	1 492	1 741	1 989	2 238	2 486	2 735	2 985	3 233	3 482
33	1 515	1 768	2 020	2 273	2 525	2 778	3 031	3 283	3 536
33.5	1 538	1 795	2 050	2 307	2 563	2 819	3 077	3 333	3 590
34	1 561	1 822	2 081	2 342	2 601	2 861	3 123	3 383	3 644
34.5	1 584	1 849	2 111	2 376	2 639	2 903	3 168	3 433	3 698
35	1 607	1 875	2 142	2 410	2 678	2 946	3 214	3 482	3 750
35.5	1 630	1 902	2 173	2 445	2 716	2 988	3 260	3 532	3 804
36	1 653	1 929	2 203	2 479	2 754	3 030	3 306	3 582	3 858
36.5	1 676	1 956	2 234	2 514	2 792	3 072	3 352	3 632	3 912
37	1 699	1 982	2 264	2 549	2 831	3 114	3 398	3 681	3 964
37.5	1 722	2 009	2 295	2 582	2 869	3 156	3 444	3 731	4 018
38	1 745	2 036	2 326	2 617	2 907	3 198	3 490	3 781	4 072
38.5	1 768	2 063	2 356	2 651	2 945	3 240	3 536	3 831	4 126
39	1 791	2 090	2 387	2 686	2 984	3 282	3 582	3 881	4 180
39.5	1 814	2 116	2 417	2 721	3 022	3 324	3 628	3 930	4 232
40	1 837	2 143	2 448	2 755	3 060	3 366	3 672	3 980	4 286
40.5	1 859	2 170	2 479	2 790	3 088	3 408	3 720	4 029	4 340
41	1 882	2 196	2 510	2 824	3 136	3 450	3 766	4 078	4 392
41.5	1 905	2 223	2 540	2 858	3 175	3 492	3 812	4 128	4 446
42	1 928	2 250	2 570	2 892	3 214	3 534	3 858	4 178	4 500
42.5	1 951	2 277	2 601	2 927	3 252	3 576	3 904	4 228	4 554
43	1 974	2 304	2 632	2 962	3 290	3 618	3 950	4 278	4 608
43.5	1 997	2 331	2 662	2 996	3 328	3 661	3 995	4 328	4 662
44	2 020	2 358	2 692	3 030	3 366	3 704	4 040	4 378	4 716
44.5	2 043	2 385	2 723	3 065	3 404	3 746	4 086	4 428	4 770
45	2 066	2 412	2 754	3 100	3 442	3 788	4 132	4 478	4 824
45.5	2 089	2 438	2 785	3 134	3 481	3 830	4 178	4 527	4 876
46	2 112	2 464	2 816	3 168	3 520	3 872	4 224	4 577	4 928
46.5	2 135	2 491	2 846	3 202	3 558	3 914	4 270	4 626	4 982
47	2 158	2 518	2 876	3 236	3 596	3 956	4 316	4 676	5 036
47.5	2 181	2 545	2 907	3 271	3 635	3 998	4 362	4 726	5 090
48	2 204	2 572	2 938	3 306	3 674	4 040	4 408	4 776	5 144
48.5	2 227	2 599	2 968	3 341	3 711	4 082	4 454	4 826	5 198
49	2 250	2 626	2 998	3 376	3 748	4 124	4 500	4 876	5 252
49.5	2 273	2 653	3 029	3 410	3 787	4 166	4 546	4 926	5 306
50	2 296	2 680	3 060	3 444	3 826	4 208	4 592	4 976	5 360
50.5	2 319	2 706	3 091	3 478	3 864	4 250	4 638	5 025	5 412
51	2 342	2 732	3 122	3 512	3 902	4 292	4 684	5 074	5 464
51.5	2 365	2 759	3 152	3 547	3 940	4 334	4 730	5 124	5 518
52	2 388	2 786	3 182	3 582	3 978	4 376	4 776	5 174	5 572
52.5	2 411	2 813	3 213	3 616	4 016	4 418	4 822	5 224	5 626
53	2 431	2 840	3 244	3 650	4 054	4 460	4 868	5 274	5 680
53.5	2 457	2 867	3 274	3 685	4 093	4 502	4 914	5 324	5 734
54	2 480	2 894	3 304	3 720	4 132	4 544	4 960	5 374	5 788
54.5	2 503	2 920	3 335	3 754	4 170	4 586	5 006	5 423	5 840
55	2 526	2 946	3 366	3 788	4 208	4 628	5 052	5 472	5 892
55.5	2 549	2 973	3 397	3 823	4 246	4 670	5 098	5 522	5 946
56	2 572	3 000	3 428	3 858	4 281	4 712	5 144	5 572	6 000
56.5	2 595	3 027	3 458	3 892	4 322	4 755	5 189	5 622	6 054
57	2 618	3 054	3 488	3 926	4 360	4 798	5 234	5 672	6 108
57.5	2 641	3 081	3 519	3 961	4 399	4 840	5 280	5 722	6 162
58	2 664	3 108	3 550	3 996	4 438	4 882	5 326	5 772	6 216
58.5	2 687	3 135	3 580	4 030	4 476	4 924	5 372	5 822	6 270
59	2 710	3 162	3 610	4 064	4 514	4 966	5 418	5 872	6 324
59.5	2 733	3 188	3 641	4 098	4 552	5 008	5 464	5 921	6 376
60	2 756	3 214	3 672	4 132	4 590	5 050	5 510	5 970	6 428

Mr. Conover.

TABLE 7.—Section Moduli of Girders.
4 Angles and Web.

4 Angles.	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″
Web, 14 Inches.									
5 × 3 1/2 × 3/8″...	74	75	77	78	80	81	83	84	86
7/16...	83	85	86	88	89	91	92	94	95
1/2 ...	92	94	95	97	98	100	101	103	104
9/16...	101	102	104	105	107	108	110	111	113
6 × 4 × 3/8″...	85	87	88	90	91	93	94	96	97
7/16...	97	98	99	101	103	104	106	107	109
1/2 ...	108	109	110	112	114	115	117	118	120
9/16...	118	120	121	123	124	126	127	129	130
Web, 16 Inches.									
5 × 3 1/2 × 3/8″...	87	89	91	93	95	97	99	101	103
7/16...	98	100	102	104	106	108	110	112	114
1/2 ...	109	111	113	115	117	119	121	123	125
9/16...	120	122	124	126	128	130	132	134	136
6 × 4 × 3/8″...	100	102	104	106	108	110	112	114	116
7/16...	114	116	118	120	122	124	126	128	130
1/2 ...	127	129	131	133	135	137	139	141	143
9/16...	139	141	143	145	147	149	151	153	155
Web, 18 Inches.									
5 × 3 1/2 × 3/8″...	101	104	106	109	111	114	117	119	122
7/16...	114	116	119	122	124	127	129	132	135
1/2 ...	126	129	131	134	137	139	142	144	147
9/16...	139	141	144	146	149	152	154	156	159
6 × 4 × 3/8″...	116	119	122	124	127	129	132	135	137
7/16...	132	135	137	140	142	145	147	150	153
1/2 ...	147	150	152	155	157	160	162	165	167
9/16...	161	164	166	169	171	174	177	179	182
6 × 6 × 3/8″...	126	129	131	134	137	140	142	145	148
7/16...	143	145	148	151	154	156	159	162	165
1/2 ...	159	162	165	167	170	173	176	179	181
9/16...	175	177	180	183	185	188	191	194	196
5/8 ...	190	193	195	198	201	204	207	209	212
11/16...	205	208	211	214	216	219	222	225	227
3/4 ...	220	223	226	228	231	234	236	239	242
Web, 20 Inches.									
5 × 3 1/2 × 3/8″...	116	119	122	125	129	132	135	138	141
7/16...	130	133	136	140	143	146	150	153	156
1/2 ...	144	147	150	154	157	160	163	167	170
9/16...	158	161	164	168	171	174	177	181	184
6 × 4 × 3/8″...	133	136	140	143	146	149	153	156	159
7/16...	151	154	157	160	163	167	170	173	176
1/2 ...	168	171	174	177	180	184	187	190	193
9/16...	184	187	190	193	196	200	203	206	210
6 × 6 × 3/8″...	145	148	152	155	158	162	165	169	172
7/16...	164	167	171	174	178	181	185	188	191
1/2 ...	183	186	190	193	196	200	203	207	210
9/16...	200	204	207	211	214	217	221	224	222
5/8 ...	218	222	225	229	232	235	239	242	245
11/16...	236	239	243	246	249	253	256	260	263
3/4 ...	253	256	259	263	266	270	273	277	280

Mr. Conover.

TABLE 7.—Section Moduli of Girders.—(*Continued.*)

4 Angles and Web.

Web, 22 Inches.

4 Angles.	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″
5 × 3 1/2 × 3/8″...	131	135	139	143	147	151	155	159	162
7/16...	147	151	155	159	162	167	170	174	178
1/2 ...	162	166	170	175	178	182	186	190	194
9/16...	178	182	186	191	194	198	202	206	209
6 × 4 × 3/8″...	150	154	158	162	166	170	174	178	182
7/16...	170	174	178	182	186	190	194	198	201
1/2 ...	189	193	197	201	205	209	213	217	220
9/16...	207	211	215	219	223	227	231	235	239
6 × 6 × 3/8″...	164	168	173	177	181	185	189	194	198
7/16...	186	190	194	198	203	207	221	215	219
1/2 ...	207	211	216	220	224	228	232	236	241
9/16...	227	231	236	240	244	248	252	256	261
5/8 ...	247	252	256	260	264	268	272	277	281
11/16...	267	271	276	280	284	288	292	296	301
3/4 ...	286	291	295	299	303	307	311	316	320

Web, 24 Inches.

4 Angles.	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″
5 × 3 1/2 × 3/8″...	146	151	156	161	165	170	175	180	185
7/16...	164	169	173	178	183	188	192	197	202
1/2 ...	182	187	191	196	201	206	211	215	220
9/16...	198	203	208	213	218	222	227	232	237
6 × 4 × 3/8″...	169	174	179	184	189	193	198	203	208
7/16...	189	194	199	204	208	213	218	223	228
1/2 ...	210	215	220	225	230	234	239	244	249
9/16...	230	235	240	245	250	254	259	264	269
6 × 6 × 3/8″...	185	190	195	200	205	210	215	220	225
7/16...	208	213	218	223	228	233	239	244	249
1/2 ...	233	238	242	247	252	257	262	267	272
9/16...	254	259	264	269	274	279	284	289	294
5/8 ...	277	282	287	292	[illegible]97	302	307	312	317
11/16...	300	304	309	314	319	324	329	334	339
3/4 ...	321	326	331	336	340	345	350	355	360
8 × 8 × 3/4″...	403	408	413	418	423	428	433	438	443

Web, 27 Inches.

4 Angles.	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″
6 × 6 × 3/8″...	217	224	230	237	243	250	256	263	269
7/16...	245	252	258	265	271	278	284	291	297
1/2....	272	279	285	292	298	305	311	318	324
9/16...	299	306	312	319	325	332	338	345	351
5/8....	325	332	338	345	351	358	364	371	377
11/16...	351	358	364	371	377	384	390	397	403
3/4....	376	383	389	396	402	409	415	422	428
13/16...	400	407	413	420	426	433	439	446	452
7/8....	424	431	437	444	450	457	463	470	476
8 × 8 × 1/2″...	339	346	352	359	365	372	378	385	391
9/16...	373	380	386	393	399	406	412	419	425
5/8....	409	416	422	429	435	442	448	455	461
11/16...	441	448	454	461	467	474	480	487	493
3/4....	473	480	486	493	499	506	512	519	525
13/16...	505	512	518	525	531	538	544	551	557
7/8....	537	544	550	557	563	570	576	583	589

Mr. Conover.

TABLE 7.—Section Moduli of Girders.—(*Continued.*)
4 Angles and Web.

Web, 30 Inches.

4 Angles.	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″
6 × 6 × 3/8″...	249	257	265	273	281	290	298	306	314
7/16...	281	289	297	305	313	322	330	338	346
1/2 ...	312	320	328	336	344	353	361	369	377
9/16...	342	350	358	366	374	383	391	399	407
5/8 ...	372	380	388	396	404	413	421	429	437
11/16...	401	409	417	425	433	442	450	458	466
3/4 ...	429	437	445	453	461	470	478	486	494
13/16...	457	465	473	481	489	498	506	514	522
7/8 ...	485	493	501	509	517	526	534	542	550
8 × 8 × 1/2″...	389	397	405	413	421	430	438	446	454
9/16...	428	436	444	452	460	469	477	485	493
5/8 ...	467	475	483	491	499	508	516	524	532
11/16...	506	514	522	530	538	547	555	563	571
3/4 ...	544	552	560	568	576	585	593	601	609
13/16...	580	588	596	604	612	621	629	637	645
7/8 ...	616	624	632	640	648	657	665	673	681

Web, 33 Inches.

4 Angles.	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″
6 × 6 × 3/8″...	284	294	304	314	324	334	344	353	363
7/16...	320	330	340	350	360	370	380	389	399
1/2 ...	354	364	374	384	394	404	414	423	433
9/16...	388	398	408	418	428	438	448	457	467
5/8 ...	421	431	441	451	461	471	481	490	500
11/16...	454	464	474	484	494	504	514	523	533
3/4 ...	486	496	506	516	526	536	546	555	565
13/16...	517	527	537	547	557	567	577	536	596
7/8 ...	547	557	567	577	587	597	607	616	626
8 × 8 × 1/2″...	441	451	461	471	481	491	501	510	520
9/16...	486	496	506	516	526	536	546	555	565
5/8 ...	531	541	551	561	571	581	591	600	610
11/16...	575	585	595	605	615	625	635	644	654
3/4 ...	617	627	637	647	657	667	677	686	696
13/16...	657	667	677	687	697	707	717	726	736
7/8 ...	697	707	717	727	737	747	757	766	776

Web, 36 Inches.

4 Angles.	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″
6 × 6 × 3/8″...	320	331	343	355	367	379	391	402	414
7/16...	358	369	381	393	405	417	429	440	452
1/2....	396	407	419	431	443	455	467	478	490
9/16...	434	445	457	469	481	493	505	516	528
5/8....	471	482	494	506	518	530	542	553	565
11/16...	508	519	531	543	555	567	579	590	602
3/4....	543	554	566	578	590	602	614	625	637
13/16...	577	588	600	612	624	636	648	659	671
7/8....	611	622	634	646	658	670	682	693	705
8 × 8 × 1/2″...	495	506	518	530	542	554	566	577	589
9/16...	545	556	568	580	592	604	616	627	639
5/8....	595	606	618	630	642	654	666	677	689
11/16...	644	655	667	679	691	703	715	726	738
3/4....	690	701	713	725	737	749	761	772	784
13/16...	736	747	759	771	783	795	807	818	830
7/8....	781	792	804	816	828	840	852	863	875

Mr. Conover.

TABLE 7.—Section Moduli of Girders.—(*Continued.*)
4 Angles and Web.

4 Angles.	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″
Web, 39 Inches.									
6 × 6 × 3/8″...	357	371	385	399	413	427	441	455	469
7/16...	399	413	427	441	455	469	483	497	511
1/2....	441	455	469	483	497	511	525	539	553
9/16...	482	496	510	524	538	552	566	580	594
5/8....	522	536	550	564	578	592	606	620	634
11/16...	562	576	590	604	618	632	646	660	674
3/4....	601	615	629	643	657	671	685	699	713
13/16...	638	652	666	680	694	708	722	736	750
7/8....	676	690	704	718	732	746	760	774	788
8 × 8 × 1/2″...	549	563	577	591	605	619	633	647	661
9/16...	604	618	632	646	660	674	688	702	716
5/8....	659	673	687	701	715	729	743	757	771
11/16...	712	726	740	754	768	782	796	810	824
3/4....	764	778	792	806	820	834	848	862	876
13/16...	816	830	844	858	872	886	900	914	928
7/8....	865	879	893	907	921	935	949	963	977
Web, 42 Inches.									
6 × 6 × 3/8″...	394	410	426	442	459	475	491	507	524
7/16...	441	457	473	489	506	522	538	554	571
1/2 ...	486	502	518	534	551	567	583	599	616
9/16...	530	546	562	578	595	611	627	643	660
5/8 ...	574	590	606	622	639	655	671	687	704
11/16...	618	634	650	666	683	699	715	731	748
3/4 ...	661	677	693	709	726	742	758	774	791
13/16...	702	718	734	750	767	783	799	815	832
7/8 ...	743	759	775	791	808	824	840	856	873
8 × 8 × 1/2″...	607	623	639	655	672	688	704	720	737
9/16...	666	682	698	714	731	747	763	779	796
5/8 ...	725	741	757	773	790	806	822	838	855
11/16. .	783	799	815	831	848	864	880	896	913
3/4 ...	841	857	873	889	906	922	938	954	971
13/16...	898	914	930	946	963	979	995	1 011	1 028
7/8 ...	953	969	985	1 001	1 018	1 034	1 050	1 066	1 083
Web, 45 Inches.									
6 × 6 × 3/8″...	433	452	470	489	508	526	545	564	583
7/16...	483	502	520	559	558	576	595	614	633
1/2 ...	532	551	569	588	607	625	641	663	682
9/16...	580	599	617	636	655	673	692	711	730
5/8 ...	628	647	665	684	703	721	740	759	778
11/16...	675	694	712	731	750	768	787	806	825
3/4 ...	721	740	758	777	796	814	833	852	871
13/16 ..	765	784	802	821	840	858	877	896	915
7/8 ...	809	828	846	865	884	902	921	940	959
8 × 8 × 1/2″...	664	683	701	720	739	757	776	795	814
9/16...	728	747	765	784	803	821	840	859	878
5/8 ..	794	813	831	850	869	887	906	925	944
11/16...	857	876	894	913	932	950	969	988	1 007
3/4 ...	918	937	955	974	993	1 011	1 030	1 049	1 068
13/16...	978	997	1 015	1 634	1 053	1 071	1 090	1 109	1 128
7/8 ...	1 038	1 057	1 075	1 094	1 113	1 131	1 150	1 169	1 188

Mr. Conover.

TABLE 7.—Section Moduli of Girders.—(*Continued.*)
4 Angles and Web.

Web, 48 Inches.

4 Angles	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″
6 × 6 × 3/8″...	473	495	516	537	559	581	602	624	645
7/16...	526	548	569	590	612	634	655	677	698
1/2 ...	578	600	621	642	664	686	707	729	750
9/16 ..	630	652	673	694	716	738	759	781	802
5/8 ...	682	704	725	746	768	790	811	833	854
11/16...	733	755	776	797	819	841	862	884	905
3/4 ...	783	805	826	847	869	891	912	934	955
13/16...	831	853	874	895	917	939	960	982	1 003
7/8 ...	878	900	921	942	964	986	1 007	1 029	1 050
8 × 8 × 1/2″...	723	745	766	787	809	831	852	874	895
9/16...	792	814	835	856	878	900	921	943	964
5/8 ...	861	883	904	925	947	969	990	1 012	1 033
11/16 ..	929	951	972	993	1 015	1 037	1 058	1 080	1 101
3/4 ...	997	1 019	1 040	1 061	1 083	1 105	1 126	1 148	1 169
13/16...	1 063	1 085	1 106	1 127	1 149	1 171	1 192	1 214	1 235
7/8 ...	1 129	1 151	1 172	1 193	1 215	1 237	1 258	1 280	1 301

Web, 51 Inches.

4 Angles	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″
6 × 6 × 3/8″...	513	538	562	586	611	636	660	684	708
7/16...	570	595	619	643	668	693	717	741	765
1/2 ...	627	652	676	700	725	750	774	798	822
9/16...	683	708	732	756	781	806	830	854	878
5/8 ...	738	763	787	811	836	861	885	909	933
11/16...	791	816	840	864	889	914	938	962	986
3/4 ...	844	869	893	917	942	967	991	1 015	1 039
13/16...	893	921	945	969	994	1 019	1 043	1 067	1 091
7/8 ...	947	972	996	1 020	1 045	1 070	1 094	1 118	1 142
8 × 8 × 1/2″...	782	807	831	855	880	905	929	953	977
9/16...	857	882	906	930	955	980	1 004	1 028	1 052
5/8 ...	931	956	980	1 004	1 029	1 054	1 078	1 102	1 126
11/16...	1 004	1 029	1 053	1 077	1 102	1 127	1 151	1 175	1 199
3/4 ...	1 076	1 101	1 125	1 149	1 174	1 199	1 223	1 247	1 271
13/16...	1 148	1 173	1 197	1 221	1 246	1 271	1 295	1 319	1 343
7/8 ...	1 217	1 242	1 266	1 290	1 315	1 340	1 364	1 388	1 412

Web, 54 Inches.

4 Angles	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″
6 × 6 × 3/8″...	557	584	612	639	667	694	722	749	777
7/16...	618	645	673	700	728	755	783	810	838
1/2....	677	704	732	759	787	814	842	869	897
9/16...	736	763	791	818	846	873	901	928	956
5/8....	795	822	850	877	905	932	960	987	1 015
11/16...	853	880	908	935	963	990	1 018	1 045	1 073
3/4....	910	937	965	992	1 020	1 047	1 075	1 102	1 130
13/16...	965	992	1 020	1 047	1 075	1 102	1 130	1 157	1 185
7/8....	1 020	1 047	1 075	1 102	1 130	1 157	1 185	1 212	1 240
8 × 8 × 1/2″...	841	871	899	926	954	981	1 009	1 036	1 064
9/16...	924	951	979	1 006	1 034	1 061	1 089	1 116	1 144
5/8....	1 004	1 031	1 059	1 086	1 114	1 141	1 169	1 196	1 224
11/16...	1 082	1 109	1 137	1 164	1 192	1 219	1 247	1 274	1 302
3/4....	1 159	1 186	1 214	1 241	1 269	1 296	1 324	1 351	1 379
13/16...	1 235	1 262	1 290	1 317	1 345	1 372	1 400	1 427	1 455
7/8....	1 310	1 337	1 365	1 392	1 420	1 447	1 475	1 502	1 530

Mr. Conover.

TABLE 7.—SECTION MODULI OF GIRDERS.—(*Continued.*)
4 Angles and Web.

4 Angles.	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″
WEB, 57 INCHES.									
6 × 6 + 3/8″...	599	630	660	690	721	752	784	814	844
7/16...	664	695	725	755	786	817	849	879	909
1/2....	728	759	789	819	850	881	913	943	973
9/16...	791	822	852	882	913	944	976	1 006	1 036
5/8....	853	884	914	944	975	1 006	1 038	1 068	1 098
11/16...	915	946	976	1 006	1 037	1 068	1 100	1 130	1 160
3/4....	975	1 006	1 036	1 066	1 097	1 128	1 160	1 190	1 220
13/16...	1 034	1 065	1 095	1 125	1 156	1 187	1 219	1 249	1 279
7/8....	1 092	1 123	1 153	1 183	1 214	1 245	1 277	1 307	1 337
8 × 8 × 1/2″...	907	938	968	998	1 029	1 060	1 092	1 122	1 152
9/16...	992	1 023	1 053	1 083	1 114	1 145	1 177	1 207	1 237
5/8....	1 077	1 108	1 138	1 168	1 199	1 230	1 262	1 292	1 322
11/16...	1 160	1 191	1 221	1 251	1 282	1 313	1 345	1 375	1 405
3/4....	1 242	1 273	1 303	1 333	1 364	1 395	1 427	1 457	1 487
13/16...	1 323	1 354	1 384	1 414	1 445	1 476	1 508	1 538	1 568
7/8....	1 402	1 433	1 463	1 493	1 524	1 555	1 587	1 617	1 647
WEB, 60 INCHES.									
6 × 6 × 3/8″...	644	678	712	746	779	813	847	881	915
7/16...	713	747	781	815	848	882	916	950	984
1/2....	781	815	849	883	916	950	984	1 018	1 052
9/16...	848	882	916	950	983	1 017	1 051	1 085	1 119
5/8....	915	949	983	1 017	1 050	1 084	1 118	1 152	1 186
11/16...	981	1 015	1 049	1 083	1 116	1 150	1 184	1 218	1 252
3/4....	1 044	1 078	1 112	1 146	1 179	1 213	1 247	1 281	1 315
13/16...	1 106	1 140	1 174	1 208	1 241	1 275	1 309	1 343	1 377
7/8....	1 168	1 202	1 236	1 270	1 303	1 337	1 371	1 405	1 439
8 × 8 × 1/2″...	971	1 006	1 041	1 076	1 111	1 146	1 181	1 216	1 251
9/16...	1 061	1 096	1 131	1 166	1 201	1 236	1 274	1 306	1 341
5/8....	1 151	1 186	1 221	1 256	1 291	1 326	1 361	1 396	1 431
11/16...	1 240	1 275	1 310	1 345	1 380	1 415	1 450	1 485	1 520
3/4....	1 328	1 363	1 398	1 433	1 468	1 503	1 538	1 573	1 608
13/16...	1 414	1 449	1 484	1 519	1 554	1 589	1 624	1 659	1 694
7/8....	1 498	1 533	1 568	1 603	1 638	1 673	1 708	1 743	1 778

Mr. Seaman.

HENRY B. SEAMAN, M. AM. SOC. C. E. (by letter).—In 1899 the writer presented to the Society a paper* on the use of the Launhardt Formula in railroad bridge design, and showed the error of its derivation and impracticability of its application. At the same time he suggested the advisability of appointing a Committee for the general revision of bridge specifications. That formula has now practically disappeared from use, and railroad bridge specifications have been gradually moulded into standard form.

It was with somewhat the same purpose in view that the present paper was prepared. The specifications presented therein embody many features which have become almost standard, and, with these as a basis, suggestions have been made for further advancement, in order that the work may be made more comprehensive and complete.

* *Transactions*, Am. Soc. C. E., Vol. XLI, p. 140.

There is no reason why a general specification should not apply to spans of any length. The principles of mechanics are universally applicable, and there is no fixed line of demarcation, either in theory or in practice, between long- and short-span bridges. The properties of materials are now well understood, and shop practice is so thoroughly organized that there is no economy in doing second-class work. Mr. Seaman.

The principal features necessary for such a broad application is the proper selection of a unit strain, to which all conditions could be equated, and the establishment of a formula for the purpose of equating these various conditions. The unit strain selected is that for which the static loads may be safely applied, and the formula is for the purpose of reducing all live-load strains to a static equivalent.

The formula proposed is empirical, and is the curve of a quarter ellipse. There appears to be no reason why a conic section should be especially applicable to these conditions, but the results obtained are fairly satisfactory, and nothing else has yet been proposed which meets better the tests which have been made. This is noted on the diagram, Plate X, presented by Mr. Bowen. Our theories upon the subject are so varied and hypothetical that confirmation must depend on practical conformity with tests.

Many formulas have been already proposed, and Mr. Cochrane has shown a number of these on the diagram, Fig. 19. It may be noticed, on that diagram, that the ellipse is generally lower than the other formulas, but Plate X shows it above the tests, and considerably above the latest formula proposed for the American Railway Engineering Association, that is, $I = \dfrac{100}{1 + \dfrac{l^2}{100}}$. The curve for that formula passes through the tests, and would naturally require a greater factor of safety, by the use of a smaller allowable unit strain, than would be necessary for the higher elliptical curve shown.

Objection has been made that our formula for static equivalent gives more than 100% increase for very short spans. If we were considering merely the impact from a rolling load, this criticism would be well founded, but that is only a small part of the increase. Unbalanced drivers and flat wheels, which cause the sharpest blows, are particularly effective on short spans. Tests on track have shown that this force exceeds 200%, at times. The subject of static equivalent is too broad to permit of hurried discussion; we will undoubtedly hear more of it in the future.

Together with the formula for static equivalent, must be considered the allowable unit strain on the material, as the two subjects are interwoven and inseparable. In the preparation of these specifications, Mr. Seaman.

the allowable unit strain was carefully considered. At first the formula, $S = 100 - \sqrt{20\ L - \frac{L^2}{100}}$, was adopted for static equivalent, and an allowable strain of 16 000 lb. per sq. in. was used to correspond. This, however, was not satisfactory, as it was not applicable to long spans; and even for short spans—constructed with modern shop workmanship—it seemed unnecessarily wasteful. It was then suggested that to increase both the formula and the allowable unit strain by 25% would produce practically the same bridge, except that there would be greater provision for counterstrain, and a slight saving in the dead-load section. The dead-load sections, however, are all increased when we provide for full live load over the entire span, which load actually seldom occurs. For an unloaded bridge, therefore, there is always surplus section.

In the specifications, two grades of steel have been provided: medium carbon and nickel steel. These may be used indiscriminately. the latter generally in long spans, and wherever the saving in weight offsets the higher cost of the material.

The question has been raised—and very naturally—as to the propriety of using an impact formula in the design of highway bridges. It must be remembered that this is not an impact formula, but rather a formula for static equivalent. To the writer's mind there is no impact on highway bridges, and very little, if any, on railroad bridges. It was for this reason that the term, "impact," was avoided. The increase in strain on railroad bridges is due principally to other causes than impact from the sudden application of rolling load, and, similarly, the increase of strain in highway bridges is due to the tendency to congestion rather than to impact. This tendency to congestion, or local intensity, increases with shorter spans, and the gradual variation from long spans may be represented by a formula; yet this need not necessarily be the same as that selected for railroad bridges. On the other hand, if, on investigation, the same formula would seem to apply, there is no objection to its use, and one formula, instead of two, simplifies the specification.

The known facts as to highway bridge loading are so variable and indefinite that extreme refinement is unnecessary, or impossible, and it is again found that the conic section will give results quite as satisfactory as any other formula that has been suggested. The only change which the writer has felt inclined to make is the possible reduction of the initial load from 80 to 60 lb. per sq. ft.

The excellent discussion by Mr. French must be carefully read to be fully appreciated. The writer has embodied several of his suggestions in the specifications.

The claim that reinforced concrete beams and slabs, when made continuous over supports, should receive credit for continuity, is well taken. This has already been permitted by the writer in his practice, and in the specifications under "Bending," in reinforced concrete, it would seem advisable to change the fourth clause to read as follows: Mr. Seaman.

"4th.—Partial continuity may be assumed in the design of beams and slabs, and the moment, $M = \frac{W L}{10}$, may be used in place of $\frac{W L}{8}$, where the conditions warrant it. In any case, however, steel reinforcement should be provided over the supports to prevent cracking."

In regard to timber structures, which have been mentioned, it has long been recognized that the heavy modern loading was endangering the short trestle spans. The 15-ft. span has practically been abandoned in favor of 12 ft. 6 in., and three heavy stringers are now commonly used where two light ones were formerly sufficient. Twenty-five years ago the writer designed trestle stringers at 1 000 lb. per sq. in. on the outer fiber. If that rule were still in use, trestles which have now two stringers at 1 400 lb. strain, would require three stringers of the same size. The full-sized tests on Georgia pine show an ultimate resistance to flexure of from 10 000 to less than 4 000 lb. per sq. in., on sound, new material. If the timber is retained in service until its strength decreases 50%—which is not unusual—the ultimate strength on old timber, of 2 000 lb. per sq. in., would be very close to the allowable strain of 1 800 lb. per sq. in., which we adopt for dead load. The remaining question, as to whether the action of unbalanced drivers, at high speed, produces the equivalent strain, as by the formula, is, of course, still uncertain, though it must be borne in mind that they give a severe blow.

For those who think the specification too conservative, the note, quoted by Mr. French, might be modified as follows: "In this table the values of static strains for timber are 50% greater than those formerly used* for miscellaneous loading without impact;" and, in order to conform more closely to the present general practice, the increase, as shown by the formula, might be reduced one-half in the case of timber. It may be that the elasticity of timber will serve to cushion the impact, but, on the other hand, it would hardly seem wise to be unduly economical with a material which is only used where it is comparatively inexpensive.

The fact that bridges which are subjected to high strain continue to render good service, is not of itself a scientific argument for

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* *Proceedings*, Fifth Annual Convention, Association of Railway Superintendents of Bridges and Buildings, October, 1895; *Proceedings*, American Railway Engineering Association, 1909, Vol. 10, p. 564.

Mr. Seaman. safety. The quality of timber to-day is no better than that of former years.

Mr. Gardiner offers some very interesting data in regard to emergency stops. In some instances this would seem to justify the old provision of 20% for sliding friction in bridge design, but when it is realized that this is only momentary, and that a large portion is taken up by the rail connections and floor system, it appears that only part of the thrust reaches the main structure.

The suggestion that either the writer's working strains are too high, or that the specifications formerly in use lead to wasteful designs, expresses exactly what the writer has wished to emphasize. The specifications in general use—except for very short spans—do lead to wasteful designs. Bridges designed for heavier loading, with correspondingly greater allowable strains, will have a much better distribution of material. The difficulty with the Quebec Bridge was not that the loads were too light, as the load was never applied. The cause of the disaster was insufficient latticing in unprecedentedly large compression members.

The clause of the specifications which permits the flanges of beams, when embedded in concrete, to receive 25% increase of strain, makes allowance for the fact that the concrete will relieve such flanges of a portion of the strain.

Although an allowable strain of 20 000 lb. per sq. in. is permitted on steel in concrete, for uniformity, as a matter of fact, it will never receive such a strain. The suggestion to use 0.7% to 1.5% of steel, limits the strain to less than 16 000 lb. per sq. in., by reason of the comparative elasticity assumed for steel and concrete.

The writer has adopted a number of the excellent suggestions made by those who have taken part in the discussion, and hereby expresses his thanks, as the spirit of discussion, in every case, shows a disposition to share the responsibility which we are mutually carrying.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 1224

RETRACEMENT-RESURVEYS—COURT DECISIONS AND FIELD PROCEDURE.*

By N. B. Sweitzer, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. W. Newbrough, J. Francis Le Baron, A. M. Strong, Leonard S. Smith, H. V. Hinckley, James L. Davis, A. T. Parsons, J. C. Carpenter, and N. B. Sweitzer.

Volumes have been written on land surveying, from the simple schoolroom treatise to the numerous and extensive works on geodetic and topographic work, as well as on geodetic astronomy, which is associated therewith. Apparently, they all deal with one idea: the accurate measurement and calculation of lines and angles. This is necessary in such surveys as are required in the construction of a railroad or canal, the erection of buildings and bridges, and also the location of the geodetic co-ordinates for a point on the earth's surface from the heavenly bodies; but there are surveys differing from these, and, thus far, they have been neglected by the textbooks; and only the rapid settling of new countries, in recent years, by an advancing civilization, has demanded a fuller treatment of retracement-resurveys. This means that the theory of surveying in all its branches, as taught in our schools and colleges, should co-ordinate itself and recognize the fact that it has to deal with a new element, "The Error," and should treat it both in a scientific and legal way. The new engineer (and also the old one, for that matter) commences to dodge as soon as he finds himself coming in contact with this; it is his *bête noire*.

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* Presented at the meeting of March 6th, 1912.

Our early surveys are a mass of errors; and, in order to deal with them intelligently, the surveyor should have an exact knowledge of mathematics, a thorough understanding of the law applicable to boundary lines and old corners, and also a personal equation of adjustment for local conditions.

In the past, many rules have been made for special cases, for retracing old and erroneous surveys, only to be found wanting when applied in general. It is also true that what will hold good for small surveys, in field location, will not be practical when large areas are under consideration.

The present method of retracement-resurvey work, which has been carried on in Nebraska for the last few years, apparently solves the problem; and the following is a synopsis thereof:

Original Surveys.

The Act creating the present system of Government surveys of the public lands was passed in 1785; and, although differing in some respects from the present original surveys, the general scheme was practically the same in regard to the establishment of a rectangular system. This Act was followed by the organization of the personnel required to execute these surveys, and the appointment of a Surveyor-General, pursuant to an Act approved May 18th, 1796.

On April 25th, 1812, another Act was passed by Congress, providing

"That there shall be established in the Department of the Treasury an office to be denominated the General Land Office, the chief officer of which shall be called the Commissioner of the General Land Office, whose duty it shall be, under the direction of the head of the Department, to superintend, execute, and perform all such acts and things touching or respecting the public lands of the United States, and other lands patented or granted by the United States."

This was followed by several Acts to promote the well-being of the service when conditions required, among others, an Act, approved May 29th, 1830 (Secs. 2412 and 2413, R. S.),

"provides for the fine and imprisonment of any person obstructing the survey of the public lands, and for the protection of surveyors, in discharge of their official duties, by the United States Marshal, with sufficient force, whenever necessary."

Another Act, approved July 4th, 1836, reorganized the General Land Office and transferred it to the Department of the Interior.

The Act approved May 30th, 1862 (Sec. 2399, R. S.), reads:

"That the printed manual of instructions relating to the public surveys, prepared at the General Land Office, and bearing the date Feb. 22, 1855, the instructions of the Commissioner of the General Land Office, and the special instructions of the Surveyor-General, when not in conflict with said printed manual or the instructions of said Commissioner, shall be taken and deemed a part of every contract for surveying the public lands of the United States."

The last manual was issued under the above authority.

Reference to further Acts of Congress, defining the public lands survey, may be found in Volumes 1, 2, 3, 4, 12, 18, and 19, for the years 1796, 1800, 1805, 1820, 1832, and 1875, respectively.

It will thus be seen how, by Acts of Congress and instructions from the Commissioner of the General Land Office, the public lands were authorized to be surveyed.

It will be well to note the following Act, of February 11th, 1805, U. S. Statutes at Large, Vol. 2, p. 313, Sec. 2396, U. S. Revised Statutes:

"All corners marked in the surveys returned by the Surveyor-General shall be established as the proper corners of the sections or quarter-sections which they were intended to designate, and corners of half- and quarter-sections not marked shall be placed as nearly as possible equidistant from those two corners which stand on the same line.

"The boundary line actually run and marked shall be established as the proper boundary line of the sections or subdivisions, for which they were intended, and the length of such lines as returned by either of the surveyors aforesaid shall be held and considered as the true length thereof. Each section, or subdivision of section, the contents whereof shall have been returned by the Surveyor-General, shall be held and considered as containing the exact quantity expressed in such return; and the half-sections and quarter-sections, the contents whereof shall not have been thus returned, shall be held and considered as containing the one-half or the one-fourth part, respectively, of the returned contents of the sections of which they may make part."

It will thus be seen that no dispute can arise in regard to the question of the permanency of the original surveys. Once made, and legally accepted, they have to remain for all time the basis for all future deeds and conveyances, the statutes thus reiterating the common law. The original corners and lines, as shown by the monuments on the ground and the original maps and plats, stand as permanent and unchangeable monuments, and, together with the accompanying field notes and the original plats, form a basis, in the case of the disappearance of the original corners, by which they may be replaced by proportioning between original corners.

Section 99 of the Act of May 18th, 1796, and R. S., 2395, among other things, provides:

"The public lands shall be divided by north and south lines run according to the true meridian, and by others crossing them at right angles, so as to form townships six miles square; * * *

"Second. The corners of the townships must be marked with progressive numbers from the beginning; each distance of a mile between such corners must be also distinctly marked with marks different from those of the corners.

"Third. The township shall be subdivided into sections, containing, as nearly as may be, six hundred and forty acres each, by running through the same, each way, parallel lines at the end of every two miles; and by making a corner on each of such lines at the end of every mile. The sections shall be numbered, respectively, beginning with the number one in the northeast section, and proceeding west and east alternately through the township with progressive numbers till the thirty-six be completed."

Sec. 100, of the Act of February 11th, 1805 (R. S., 2396), among other things, provides:

"The boundaries and contents of the several sections, half-sections and quarter-sections of the public lands, shall be ascertained in conformity with the following principles:

* * * * * * *

"Third. Each section or subdivision of section, the contents whereof have been returned, by the surveyor-general, shall be held and considered as containing the exact quantity expressed in such return; and the half-sections and quarter-sections, the contents whereof shall not have been thus returned, shall be held and considered as containing the one-half or the one-fourth part, respectively, of the returned contents of the section of which they may make part."

From this it will be seen that no authority is given to make any more or less than thirty-six sections, containing, "as nearly as may be, six hundred and forty acres each," to a township, or to make any other form of subdivision. However, this is qualified for a special case, by order of the President, as where land borders on rivers, lakes, and bayous. (R. S., 2396, Sec. 102.)

It is apparent that persons wishing to obtain Government lands file according to the original Government survey, as prescribed by the above law, the plats on file in the local land office and the boundaries of such lands as located by the original Government corners on the ground, and not according to a valley, stream, or other physical condition of the terrane. Now, until these laws are repealed, no more than thirty-six sections, consisting, "as nearly as may be" of "six hundred and forty acres each," can be numbered in a township; and any resurvey, in order to be legal, will have to relocate the original Government corners in their original positions.

Resurveys.

Justice Cooley, of the Supreme Court of Michigan, has written a most valuable article on "The Judicial Functions of Surveyors," which has been used by surveyors and engineers for years as one of their guides in retracing old lines. The general principles laid down are accepted without question as good law. However, Justice Cooley contemplated only the wooded districts of Michigan and adjoining States, where only comparatively small areas were in question, and where the footsteps of the original surveyor could be located positively, at no very great distance, by the remains of old bearing trees, permanent lakes, and well-cut banks of streams and rivers. He did not contemplate the great areas of land on the Western prairies, unmarked by a tree, uncut by lakes or streams—an undulating terrane where one square mile is a facsimile of the other, and where no natural object distinguishes one mile from another. The original field notes, for mile after mile, often read "40.00" and "80.00" chains, "over rolling sand hills." Nor did he contemplate the fact that vast areas in this region had never been surveyed on the ground, but had been platted and the plats placed on file, giving varying distances and areas to be filed on by settlers, and they would claim, on the ground, the distances and areas designated by the plats on file. Here, then, was a vast checker-board, as represented by the plats on file, with innumerable squares of varying areas, which should have corresponding areas on the ground. Obviously, if one of these checks were accidentally or wilfully misplaced, the whole scheme would be disturbed. A study of the situation will reveal the fact that to be harmonious with, and to fulfill the requirements of, the Court decisions, every claimant, where no original corners exist, is entitled to his lands as designated by the plat thereof, and, if the area on the ground covered by the platted areas contains more land than shown, he is entitled to his proportionate share. Otherwise, the harmony existing between plats and terrane will be disturbed, and some one not entitled to it will secure more than his just share. Likewise, a deficiency will cause an unjust loss of land.

Keeping the justice and equity of the foregoing always in mind, it is also well to remember that adverse possession does not run against the Government.

The following Court decisions in regard to resurveys are quoted from the works of F. Hodgman, Justice Cooley, and the late J. B. Johnson, M. Am. Soc. C. E.

"When boundaries mentioned are inconsistent with each other, those are used which best show the intention manifest on the face of the deed." (Gates *v.* Lewis, 7 Vt., 511.)

"Where one part of the description in a deed is false and impossible, but by rejecting that a perfect description remains, such false and impossible part should be rejected." (Anderson *v.* Baughman, 7 Mich., 79; Johnson *v.* Scott, 11 *id.,* 232.)

"Where boundaries of lands are fixed, known, and unquestionable monuments, though neither courses, distances, nor computed contents correspond, the monuments must govern." (Perman *v.* Wead, 6 Mass., 131; Nelson *v.* Hall, 1 McLean, U. S., 518.)

"Marked lines and corners control course and distance. Surplus lands do not vitiate a survey, nor does a deficiency of acres called for in a survey operate against it. Whenever the boundaries can be established, they must prevail." (Robinson *v.* Moore, 4 McLean, U. S. C. C., 279; Marrow *v.* Whitney, 5 Otto, U. S., 551.)

"A survey must be closed in some way or other. If this can only be done by following the course the proper distance, then it would seem that distance should prevail; but when the distance falls short of closing, and the course will do it, the reason for observing distance fails." (Doe *v.* King, 3 How., Miss., 125.)

"A line actually marked must be adhered to, though not a right line from corner to corner. When a line has been marked only part of the way, the remainder of the line must run direct to the corner called for." (Cowan *v.* Perkins, 2 Jones Law Rep., N. Y., 222.)

"Of two overlapping surveys, the one first made has priority, particularly where the second is bounded with express reference to the first." (Van Amburgh *v.* Hitt, Mo. Sup. 22 S. N. W., 177.)

"The beginning corner of a survey, as given in the field notes, is of no more dignity than any other corner found on the ground." (Cox *v.* Finks, Tex. Civ. App., 41 S. W., 95.)

"Where original surveys *have been made,* and returned as a block into the land office, the location of each tract therein may be proved by proving the location of the block. Every mark on the ground tending to show the location of any tract in the block, is some evidence of the location of the whole block, and therefore of each tract." (Coal Co. *v.* Clement, 95 Pa. St., 126.)

"Where it is doubtful which of two lines of monuments is the true government line, other things being equal, that one is to be so taken which most nearly conforms to the field notes." (Hubbard *v.* Dussy, 22 Cal., p. 214.)

"Where a boundary line is assented to by the owner of a tract of land at the time when there was no dispute concerning such line, and on the supposition that it is the true boundary, he is not estopped, on discovering that such is not the case, from claiming title to the true boundary." (Schraeder Min. & Mfg. Co. *v.* Packer, 9 S. St., 385.)

"A county surveyor, employed to restore the lines and corners of adjoining tracts of land, according to the original government survey, found township corners only, then (the other quarter and section corners being missing), ran a straight line from one township corner to the other, and on this line placed the quarter and section corners, but did not take any testimony to ascertain the lines or corners of the original survey, did not attempt to prove his lines or corners by re-establishing the missing corners from all the known original corners, in all directions, did not sufficiently regard the field notes of the original survey, and did not, where the original monuments had disappeared, regard the boundary lines long recognized and acquiesced in. Held: that such a survey is incomplete, and cannot be approved as the true and correct determination of the boundaries and corners, as originally established by the Government." (Reinert *v.* Brunt, 21 Kan., p. 807.)

"Lands sold under the U. S. surveys pass according to the description of the legal subdivisions, whether these subdivisions contain the legal quantity or not, more or less." (Fulton *v.* Doe, 6 Miss., 751.)

"Quarter posts of the Government survey are to be as much respected as the corners of townships or sections, however distant from the center line." (Comphall *v.* Clark, 8 Mo., 558.)

"If the distance between recognized Government corners, as originally established, over-runs or under-runs that given in the field notes, it should be divided *pro rata* between the intervening sections. The original field notes should be the main guide. Section corners being often deflected, the true corners must be tested by east and west distances from the recognized Government corners yet standing in the same township, as well as by north and south distances." (Martz *v.* Williams, 67 Ill., 306.)

"Unknown corners must be found by the corroborative testimony of all known corners with as little departure as may be from the system adopted on the original surveys, without giving preponderance to the testimony of any one monument above another.

"In re-establishing lost corners between remote corners of the same survey, when the whole length of the line is found to vary from the length called for, we are not permitted to presume that the variance arose from the defective survey of any part, but must consider, in the absence of circumstances showing the contrary, that it arose from the imperfect measurement of the whole line, and distribute such variance between the several subdivisions of the whole line in proportion to their respective lengths." (Moreland *v.* Clark, 8 Mo., 556.)

"The rule of common sense and of law is that the surplus or deficiency is to be apportioned between the lots, on the assumption that the error extends alike to all parts." (O'Brien *v.* McGrane, 29 His. Reports, 446; Quinnin *v.* Reizers, 46 Mich., 605.)

The early surveys of these Western plains were naturally hurried, from the nature of the case—contract work. The object of the surveyors was to run the greatest number of miles in the shortest space of time. Therefore, to them, time occupied in building corners was so much loss, and, as a consequence, the original corners were mostly of the minimum size, and sometimes of irregular shapes. Time and the elements soon apparently obliterated these corners. Whole townships and counties were thus affected, and, to make matters more complicated, when final occupation commenced, a vain search for corners was begun by, and unending lawsuits ensued among, the settlers and owners of these lands. In certain instances, some of the early contractors had neglected to set the interior section corners, and in other instances, even the town and range lines, or a check, were never surveyed, but, making bad conditions worse, false field notes were placed on file.

Here, indeed, was a complicated problem for adjustment. Note what the Courts say about resurveys, and how many questions of law and of a technical surveying nature are involved.

Fig. 1.—Surveying Parties, Garden County, Nebraska, May 1st, 1911.

Fig. 2.—Surveyors' Camp, Garden County, Nebraska, May 1st, 1911. 2 Ft. of Snow on Level.

Fig. 3.—Outline of East Pit of Original Standard ¼ Corner, T. 25 N., R. 22 W., 6th P. M., S. 36, Nebraska, in Heavy Soil. Corner About 40 Years Old; All Surface Indications Obliterated.

A few years ago an attempt was made to ignore the early surveys, and other new surveys were made on top of the original ones, running straight lines without relation to the original corners. All went well for a few years, but the inevitable happened. For example: A filed on the original plat for Section 1; B filed on the new plat of an adjoining section. These did not agree, and, therefore, A and B went to the Courts. A township is like a checker-board—disturb one square and the others will have to be adjusted. The dispute of A and B disturbed the remainder of the township; and then came the search for original corners, as the Courts had always decreed. Land held, patented, and located on the ground, by the original corners, and taken in accordance with its accompanying map on file at the local land office, cannot easily be disturbed, when the claimant is aware of his rights, for our common law and the decisions above quoted, give him indisputable possession, for his patent is sacred; and such it should be; otherwise, no one would be secure in the possession of his holding, whether it were a dugout on a bleak prairie or the uncounted acres of some rich cattle baron.

Surveyors and others should remember the following, when attempting novel methods of surveying:

"All corners marked in the surveys returned by the Surveyor-General shall be established as the proper corners of the sections or quarter-sections, * * * and the length of such lines as returned * * * shall be held and considered as the true length thereof." (U. S. Revised Statutes.)

After reviewing the above, we are forced to recognize the following facts: Corners legally established by the Government remain fixed and cannot be changed, no matter how erroneous they may be. This, of course, refers to the Public Land Surveys, and affects land titles. The more we step and consider how loosely these old surveys were made and how little attention was formerly given to them, either by those in power or by the people in general, the more we are surprised. Other great surveying departments of the Government, such as the Coast and Geodetic Survey, have always used the most refined methods, have had the best of personnel, and have taken ample time to measure and calculate lines and angles properly. Yet, when they find errors in their monuments or corners, no law intervenes to prevent them from correcting them and changing the corners. Corners of Public Land Surveys, however, cannot be changed, and gross errors can never be eliminated. Now, under these conditions, it is not surprising that so little care was taken in the early public land surveys. Time and results have shown lack of care, and much of the blame should be laid on the contract system and rapid and cheap work. We certainly have had a sufficient amount of truly cheap original surveys, which, in the end, will entail far more expense than the original cost, in litigation and money, both to the Government and to the citizens affected thereby. In addition, they will check materially the development of the West, as permanent improvements cannot be placed on lands which are constantly threatened with lawsuits.

Original Monuments.

As this has to do with the prairie regions, a description of bearing trees is hardly necessary, but, in hunting for these accessories to original corners, it might be well to give a short explanation of how they appear years after they are marked.

After having established his corner, the Government surveyor oriented his instrument above it, and, selecting such trees as were required, took their bearing and distance. His assistants, in the meantime, made a cut about 4 in. wide and 30 in. long directly above the roots, with another blaze below this, 4 by 4 in. On the upper blaze was marked with a "scribe" the township, range and section; on the lower the mark, "B. T." Nature eventually healed this wound, and finally it barked over, no trace of the blaze remaining. However, the practiced eye can easily discern the indications of the axe mark in the bark above or below where the blaze commenced or ended. The new bark may be removed without defacing the old mark, by cutting, above and below where the old blaze started, to the depth of the old wound; then, with a boxing blow of the axe, the new growth will come off, leaving the letters, cut by the scribe years before, as plain as the day they were made. Furthermore, the piece which was removed will have the same letters in relief, and may be carried away and used as evidence, leaving Nature to repeat the same process of preserving the marks. The year in which the mark was made may be ascertained by the rings on the tree, and can be checked by the original record.

At first thought, one would suppose that the bearing tree was the only lasting natural accessory to perpetuate the original corner, and that the open prairies were deficient in that respect.

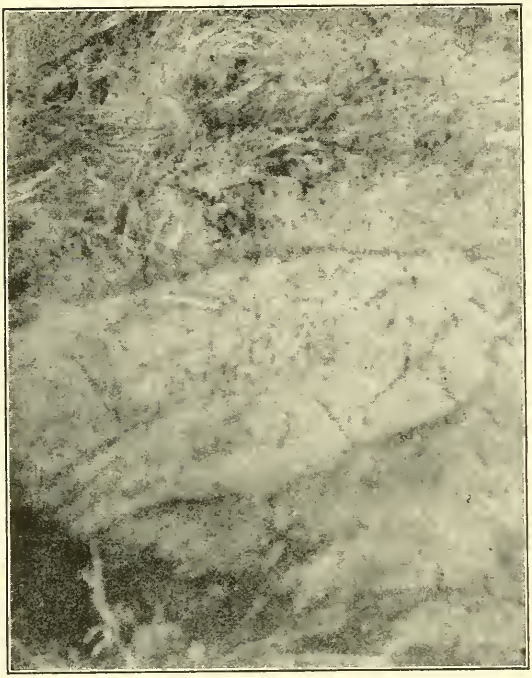

FIG. 4.—OUTLINE OF EAST PIT OF ORIGINAL SECTION CORNER, SECTIONS 35-36, S. C., T. 25 N., R. 16 W., 6TH P. M., NEBRASKA. IN SANDY SOIL AND HIGH GRASS. ORIGINAL CORNER TRAMPLED BY CATTLE. NO SURFACE INDICATIONS. AGE ABOUT 40 YEARS.

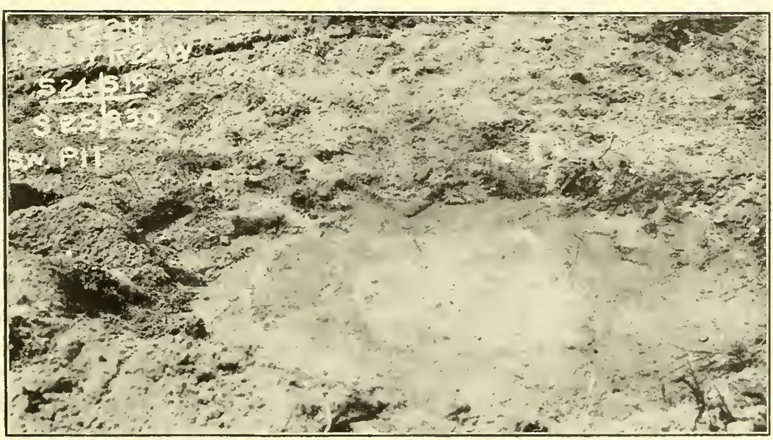

FIG. 5.—OUTLINE OF PIT OF ORIGINAL GOVERNMENT SURVEY, NEBRASKA, 40 YEARS OLD. NO SURFACE INDICATIONS, AND IN SANDY SOIL. MARKINGS ON BRASS CAP OF NEW IRON CORNER SHOWN IN UPPER LEFT CORNER.

Such, however, is not the case. The corners made in the original surveys on the open plains, where there were no rocks or stones, consisted usually of a mound of earth about 4 or 5 ft. in diameter at the base, 3 ft. high, with four pits, 18 by 18 by 12 in., north, south, east, and west, or northeast, southwest, etc., about 5 ft. from the mound.* The notes mention a deposit of charcoal, but, in practice this was very seldom made. Early surveyors like to tell the story that, as the regulations required the "deposit of a quart of charcoal," a quart of charcoal, well wrapped up in a canvas bag attached to a stout string, was actually deposited in the hole beneath the corner, with appropriate ceremonies, and immediately withdrawn, this same quart of charcoal doing duty for thousands of corners, in fact for a whole season's work. Stakes and posts were more often used. Their ends were frequently charred, and this was sometimes supposed to do duty for the charcoal.

Time has passed since these corners were originally made. The fierce winds of spring and fall, fanning the prairie fires, rapidly filled these pits with black ash and charcoal of burnt grasses and weeds, surface soils, and sands foreign in nature and color to their sides and bottoms, and soon leveled them, as well as the mounds of fresh earth. After the winter snows had gone and spring arrived, grasses and weeds quickly sprang up, sodding over both pits and mound, and obliterating, to all appearances, the boundaries of some one's home. These marks, however, were not really destroyed, for, with a light mould-board breaking plow, the sod may be turned back, and, as it curls up, a discoloration different from the prevailing color of the sod appears. Then, with spades sharpened to a beveled edge by files carried for that purpose, these discolorations may be "shaved" and the outlines of an original pit soon appear, the charred grasses and foreign soils and sands, which filled it, showing plainly in texture and color against the homogeneous soil of its sides. The remaining pits may soon be disclosed, and the original corner re-established and perpetuated by the cement and brass-capped monument now used by the Land Office.

Thus, after all, the method of finding the original marks on the

* In regard to the direction of the pits, in the original corners, from the mound, no set rule seems to have been followed in the early surveys. In the same contract and township, pits were placed sometimes north and south of the mound, and sometimes east and west. Their sizes also vary.

bearing tree in the forest is practically the same as locating the corresponding corner on the treeless plains of the West.

Having reviewed some of the Court decisions affecting resurveys and indicated the method of searching for an original corner in the forest and on the plains, the methods of field procedure will be described.

A Township Originally Partly Surveyed.

It will be assumed, first, that a township has only been partly surveyed by the deputy surveyor, or that only a portion of his original corners can be found.

First, make a resurvey of the exteriors; find all original corners possible, and proportion in those corners where the original ones cannot be found. Make a closed survey; then find the length and bearing of a line starting from the center of the south town line, or corner to Sections 33 and 34, which will intersect the corner on the north town line, or the corner to Sections 3 and 4, or conversely, or the line nearly at right angles about 6 miles long east and west through the township. The former is preferable as convergency does not enter into the field work. Run this line, searching for original corners, and placing temporary mile and half-mile points, locating all original corners by rectangular co-ordinates to this line. When the original corners are found, tie them to each other and the exteriors by true course and distance, calculated by this line; continue this process through the township north and south, east and west, until all original corners are found. It will be apparent, from the foregoing method, that the original surveyor, starting from any corner on one side of a township, no matter how he wandered in setting his corners in the central part of the township, must ultimately arrive at the corner on the opposite side. Therefore, having the calculated course and distance, one has a definite line to run, and must sometime cross his course, or at least arrive at the same corner, and thus have a better chance to find his marks, than any haphazard interior line which may be guessed at as a random.

Now, assume that the southwest nine sections have not been surveyed, or that the original corners cannot be found. One has the 3 miles of township exterior from the southwest corner of the township north and the 3 miles east, and the course and distance between the interior section corners, north and south, east and west, opposite these lines. Make a complete closure of this area; calculate the distances from the interior section corners to their opposite corners, in this area, on the town and range lines. After these distances have been secured, taking the original plat, proportion in the 3 miles north, south, east, and west. Now, having the courses and distances of the perimeter of this area, and the proportional distances of every mile of its interior, the courses and distances of all interior miles can be calculated. This will furnish all necessary data to locate on the ground all missing corners in this portion of the township.

A Township Never Originally Surveyed.

As in the foregoing illustration, find all original corners on the exterior of the township and relocate all missing exterior corners by proportional measurements from the original plat; then proceed to search for original corners on the interior, as explained in the previous case. Having ascertained that none exist, or that the township was never originally surveyed, proceed as follows: Calculate the distances through the township on section lines north, south, east, and west. Then, taking the original map as a basis, give each mile in the township its proper proportional distance. (Always check a given north area against a south area, an east area against a west area. They will always check if the exteriors are an accurate closure.) Now, having all interior lines properly proportioned, the courses are thus secured: Take the center line north and south, the line between Sections 33 and 34; take the sum of the south boundaries of Sections 34, 35, and 36, the course of the east boundary of Section 36 (algebraically), and the sum of the north boundaries of the above sections; then the difference of the easting and westing, not neglecting curvature, will give the course sought. Then, check to the west the north and south boundaries of Sections 33, 32, and 31, with the course of the west boundary of Section 31, which should give the same result as that secured above. The remaining 5 miles north to the corner of Sections 3 and 4 may be secured in the same way, checking east against west, the courses of all north and south lines being secured in the same manner. The east and west miles are secured by the same method. The course of the line between Sections 13 and 24 is the difference of the east and west boundaries of Sections 1, 12, and 13; get the course of the north boundary of Section 1; then the difference of northing and southing will give the course of the line between Sections 13 and 24. Check by the difference of northing and southing of Sections 24, 25, and 36, and the south boundary of Section 36. The lines between Sections 14 and 23, 15 and 22, 16 and 21, 17 and 20, 18 and 19 are secured in the same manner, as are the remaining east and west miles.

Having, then, the data to place it instrumentally on the ground, commence, if possible, at the corner to Sections 33 and 34 on the township line. Run the calculated courses and distances as a random line to the corner of Sections 3 and 4, on the opposite town line, setting temporary corners at the calculated corner and quarter-corner positions, and correct back on the true line. Then start, if possible, at the corner to Sections 13 and 24 on the east range line, and run the calculated courses and distances as a random line to the corner to Sections 18 and 19 on the west range line, setting temporary points as above, and correcting back. This will locate the center of the township, and the remaining quarters can be located as above.

Now, this fulfills the requirements of the Court decisions, giving all claimants their proportional part of all excesses, and also divides proportionally all deficiencies, as by this method no excesses are carried into any part of the township.

Checks between standards and parallels affecting the town and range lines are effected in the same manner. Always work from the center toward the exteriors, having first made a closed survey of the exteriors. The method of differences will shorten the proportional method when the differences are not too great.

Instruments.

The original surveys were made, as a rule, with the solar. This instrument, in the hands of a competent instrument man, gives good results, especially in timber and brush, where many sights are taken in a mile. When in adjustment, it also gives a good approximate meridian, and, as used by the average transitman, in a hurried set up, when many sights are used, the error, if any, tends to counterbalance. In an open country, however, the tendency is to take a few hurried solar observations, where long sights can be easily secured, and then resume the tangent lines. This gives a ragged line; and long experience has shown that, while sometimes good closures result, many of the courses which cause trouble with interior section closures are not correct, but tend to balance in the final result. This being the case, only transit lines should be used; and a light mountain transit is advised. Observations on Polaris or any circumpolar star are most readily made, and an elongation observation or an approved daylight hour angle or two should be taken on every township exterior, and one for the interiors. All lines should be back and fore transit lines, double-centered, when necessary.

The chains should be tested every two or three days, and, if a tape is used, care should be taken to see that the handles are not sprung.

Transits should always be in adjustment, as the time required to keep them so is only a few minutes, and is much less than necessary with a solar instrument.

Observations.

The following observations should be used on circumpolar stars, and on equatorial stars only when the situation demands.

The elongation observation will not be given in detail, as the procedure is so simple that it seems unnecessary. It might be pertinent, however, to state that the instrument should be in perfect adjustment, especially the levels. Always take direct and reverse pointings at the star. Place the mark on the sky line, about 30 chains distant, so that, if possible, it is east of the star for east elongation and west of the star for west elongation. Use an electric flash-light for the cross-hairs and to read the verniers. Read both verniers. The surveyor should not wear a long coat which the wind will blow against the tripod legs while he is taking an observation.

To compute the azimuth of a circumpolar star at elongation, use the equation:

$$\sin. A = \sec. \phi \cos. d.$$

To know the time of elongation, use the following:

$$\cos. te = \tan. \phi \cot. d.$$

The hour angle, added to the right ascension, equals the sidereal time of the star's elongation; reducing to mean time, equals the local mean time sought.

The "American Ephemeris" should be consulted for right ascension and declination.*

* See "Fixed Stars, Constants of Struve and Peters." For elaborate methods of azimuth at elongation, see "Geodetic Astronomy" (John F. Hayford, M. Am. Soc. C. E.), George C. Comstock, W. W. Campbell, and others.

The most convenient form of observation, and the one that will appeal to the engineer most, on account of its convenience, is that by the daylight hour angle.

After running on a tangent all day, the writer, just before sundown, as the wagons were swinging into camp, has often set up in front of the place where his tent was to be pitched, and, using a mark on the sky line perhaps a mile away, observed Polaris, securing at least two direct and two reversed observations. The whole series usually does not take more than 15 min., the sun being still many minutes from the horizon. This is entirely a daylight operation, no lamps being required to illuminate the cross-hairs and verniers, hence there is little chance of error from this source.

The operation is very simple. Having the tangent, as assumed above, the north point is easily approximated. The position of Polaris for that date is calculated roughly (either for upper culmination or elongation), and its position relative to the celestial pole is found. Then, orienting the instrument on the meridian, fix the telescope on sidereal focus, point the line of collimation to the celestial pole by means of the vertical arc, then raise or lower it according as the star is above or below the elongation points, and move it east or west according as the star is east or west of the celestial pole. If it is required to be exact, use the following:

$$\text{Tan. } A = \frac{\sin. t}{\cos. \phi \tan. d - \sin. \phi \cos. t}$$

which is the azimuth of Polaris at any hour angle. In this and for nearly all observations, every practicing engineer and surveyor should have the pamphlet of the General Land Office on the azimuths of Polaris, in which this is all tabulated. It takes but a moment to secure the necessary functions of the star and the azimuth required for the line check. One should not be discouraged if the star is not found at once. When the eye does discover it, it looks like an electric bulb.

Taking advantage of a clear evening, on May 1st, 1907, the writer secured the correct time by telephone from the Western Union telegraph office, and found that his watch (Waltham Vanguard, No. 11 000 844) was 2 min. slow. Then the following daylight observations were made on *Alpha Ursæ Minoris.* No lights were required, as the cross-hairs and verniers were plainly visible. A mark was set on the sky-line 30 chains north, and slightly east, as shown by the subsequent observation. The levels were in perfect adjustment, the bubbles remaining in the center of the tubes continually.

May 1, 1907; Lat. 42° 33′ N.; Long. 101° W. Gr.

Watch slow 2 min. of Standard time, 105° Mer.

Angle, Azimuth mark to star,

	Time.		*Direct.*	
	h. m.	A	B	Angle.
	6 55	9° 56′	189° 56′	9° 56′
	7 00	9° 55′	189° 55′	9° 55′
			Reversed.	
	7 10	9° 53′	189° 53′	9° 53′
	7 13	9° 53′	189° 53′	9° 53′
Mean =	7 04.5			9° 54.5′
Watch correction...............	2.0			
Correct astron. time, 105° Mer..	7 06.5			
Reduced to 101° Mer...........	16.0			
Local mean time (astron.)......	7 22.5			
	24 00	h. m.		
Time of obs., Apl. 30, 1907.....	31 22.5	31 22.5		
		22 51.5		
Hour angle =		8 31.0		
U. C. Polaris, May 1, 1907.................		22 47.6		
		3.9		
U. C. Polaris, Apl. 30, 1907....		22 51.5		

Azimuth, Polaris at obs. = 1° 15′ E.

9° 54.5′
1° 15.0′
8° 39.5′ = Meridian W. of mark.

A. M. and P. M. equal altitude observations of the sun, check the meridian. Frequently circumpolar stars cannot be caught on account of local conditions. Then one has to observe equatorial stars; and, in some cases, as the sun, the following is an example:

During the execution of Government surveys in the Olympic Mountains in 1898, for many weeks at a time, circumpolar stars were invisible, on account of the Sound and Pacific fogs, the deep, narrow cañons, and the immense trees. The only heavenly body obtainable for azimuth in that region, with any constancy, therefore, is the sun, and its apparently somewhat erratic path through the heavens, as compared with the fixed celestial bodies, was more difficult to calculate than that of a star. The writer devised the following method of equal altitudes of the sun for azimuth, and this has since been adopted by the U. S. Government for like conditions. It is entirely novel, nothing like it appearing in any textbook on engineering. The following is an example:

On May 2d, 1907, at 8.30 A. M., the writer observed the sun with a Young and Sons light mountain transit, No. 7 598, with solar attachment, setting off 15° 10′ 30″ on the declination arc, and 42° 33′ on the latitude arc, thus securing a solar meridian, which was checked as follows, for a true meridian, by equal altitudes of the sun for the two astronomical triangles, A. M. and P. M.: the azimuth mark 30.00 chains north, as given by the solar observation; altitude and azimuth of the sun corrected for refraction and semidiameter; angles taken to the nearest minute; standard time, 105th Meridian.

Lat. ϕ 42° 33′. Long. λ 101° W. Gr. Decl. δ 9 A. M. 15° 10′.

$$\text{Tan.}^2 \tfrac{1}{2} A = \frac{\sin.(S - \text{Co-Alt.}) \ \sin.(S - \text{Co-Lat.})}{\sin. S \ \sin.(S - \text{Co-Decl.})}$$

A. M. triangle.

Alt. = 42° 49′

t = 90° 00′

Co-Lat. 47° 27′	84° 44′	84° 44′	84° 44′
Co-Alt. 47° 11′	47° 11′	74° 50′	47° 27′
Co-Decl. 74° 50′	37° 33′	9° 54′	37° 17′
2) 168° 88′			
84° 44′ = S			

Log. sin. 37° 33′ =	9.784941	
" " 37° 17′ =	9.782298	
	19.567239	19.567239
Log. sin. 84° 44′ =	9.998163	
" " 9° 54′ =	9.235349	
	19.233512	19.233512
Log. Tan.^{2} $\frac{1}{2}A$		2)0.333727
" " $\frac{1}{2}A$		0.166863
$\frac{1}{2}A$ =		55° 44′ 45″
A =		111° 29′ 30″

$$\left.\begin{array}{l}\text{Alt.} = 42^\circ\ 49' \\ t = 90^\circ\ 00' \\ \text{Decl.} = 15^\circ\ 14'\end{array}\right\}\text{P. M. triangle.}$$

Co-Lat. 47° 27′	84° 42′	84° 42′	84° 42′
Co-Alt. 47° 11′	47° 27′	74° 46′	47° 11′
Co-Decl. 74° 46′	37° 15′	9° 56′	37° 31′
2) 168° 84′			
84° 42′ = S			

Log. sin. 37° 31′ =	9.784612		
" " 37° 15′ =	9.781966		
	19.566578	19.566578	
Log. sin. 84° 42′ =	9.998139		
" " 9° 56′ =	9.236795		
	19.234934	19.234934	
Log. tan.^{2} $\frac{1}{2}A$		2)0.331644	
" " $\frac{1}{2}A$		0.165822	
$\frac{1}{2}A$		55° 40′ 55″	
A		111° 21′ 50″	
		111° 21.83′	

The A. M. triangle gives an azimuth of 111° 29.5′
The P. M. " " " " " 111° 21.83′

$2\,dA =$ 7.67′

Azimuth of astronomical triangles,
taken from N. $dA =$ 3.83′

As a check on the foregoing equal altitude observations, the writer uses the following:

$$dA = \frac{\frac{1}{2}(\delta - \delta)}{\cos.\ \phi\ \sin.\ \frac{1}{2}(t + t)}$$

$\frac{1}{2}(\delta - \delta) =$ Log.	0.301030
Cos. ϕ 42° 33′ "	9.867283
sin. $\frac{1}{2}(t+t)$ 45° "	9.849485
	19.716768
	0.584262

$dA =$ nat. 3.83′; dA to be turned from bisection of angle (360° — 222° 51.33′) from south to east.

Standard Parallels.

After the observations for meridian, the most important thing is to project the line thus secured. The meridian projected north and south, of course, needs no introduction; but a projection of the meridian as a true east and west line does. Books on surveying contain many tables and interesting and complicated formulas in reference to the methods of projecting a parallel of latitude with a transit instrument. Most of them make interesting mathematical problems for the office, but, for the locating engineer and his assistants, the interest ceases when field work commences.

The tangent and secant methods are the ones most used. The trouble with the tangent method is that it departs from the parallel as the square of the distance increases and soon the transit line is so far from the objective that the topography materially differs, and in timber country the cutting is not on the true line. The secant method is complicated. It first starts south of the parallel, then is north of it, and, then, again is south, in 6 miles. This means that the offsets are first made north, then south, and then again north. As most of these offsets are made by assistants, this method requires a higher standard of efficiency than is generally met by the average flagman and chainman, and is conducive to error.

To avoid the deficiencies of these methods, the writer has been using the following for some years past, and finds it practical. An example will illustrate:

At the corner to Sections 31 and 36, on the 4th Standard Parallel North, 6th P. M., Nebraska, between Ranges 32 and 33, West, after observing *α Ursæ Minoris* in the usual manner, the line was run east on the 4th Standard Parallel 40 chains, 0 links north to parallel; 1 mile, 1 link north to parallel; 1½ miles, 2 links north to parallel; 2 miles, 3 links north to parallel; 2½ miles, 5 links north to parallel; 3 miles, 8 links north to parallel. At this point the writer turned a deflection angle of 4½′ north, and proceeded as above, and at the 3-mile point arrived at the corner 6 miles east of where he started, and his course there was due east.

In this method the assistants have only to remember 3 miles of offsets, as the last 3 miles in a township are the same as the first and decreasing; and, if the offsets are forgotten, they are easily calculated by remembering that they vary as the square of the distances in miles.

Table 1 gives the offsets, in links, from the tangent to the parallel:

TABLE 1.—TRAVERSE-TANGENT: OFFSETS, IN LINKS, FROM TANGENT TO PARALLEL.

Latitude.	Miles.						Deflection angle.	Tangential angle.
	½	1	1½	2	2½	3		
35°	0	1	2	3	4	6	3′ 38″	1′ 49″
36°	0	1	2	3	5	7	3′ 46″	1′ 53″
37°	0	1	2	3	5	7	3′ 55″	1′ 57″
38°	0	1	2	3	5	7	4′ 04″	2′ 02″
39°	0	1	2	3	5	7	4′ 13″	2′ 06″
40°	0	1	2	3	5	8	4′ 22″	2′ 11″
41°	0	1	2	4	5	8	4′ 31″	2′ 15″
42°	0	1	2	4	6	8	4′ 41″	2′ 20″
43°	0	1	2	4	6	8	4′ 51″	2′ 25″
44°	0	1	2	4	6	9	5′ 01″	2′ 30″
45°	0	1	2	4	6	9	5′ 12″	2′ 36″
46°	0	1	2	4	7	9	5′ 23″	2′ 41″
47°	0	1	2	4	7	10	5′ 34″	2′ 47″
48°	0	1	3	4	7	10	5′ 46″	2′ 53″
49°	0	1	3	5	7	10	5′ 59″	2′ 59″
	5½	5	4½	4	3½	3		

This table can be used for any angle from the east and west cardinals by multiplying the deflection angle by the cosine of the course and the corresponding offsets by the square of the cosine of that course.

Table 2 has been compiled for the use of field engineers and surveyors, to enable them to make use of the different formulas for solving for azimuth, latitude, hour angle, and different functions of Polaris, without consulting so many books and tables. Sidereal time of Greenwich, mean noon, is given for reducing sidereal time to mean time, and conversely. The formulas for azimuth, time of elongation and azimuth of Polaris, at any hour angle, as well as the formulas for equal altitudes of the sun, are previously given. Mean time is the interval after mean noon. Convert this interval into the equivalent sidereal interval and add to the sidereal time of noon. Sidereal time, noon, is equal to the right ascension of the mean sun at that instant. 9.8565 s. multiplied by the longitude of the observer and applied to Greenwich sidereal time of mean noon (see Table 2), plus when west, will give the local sidereal time of mean noon, or right ascension of mean sun of observer.

TABLE 2.—*Alpha Ursæ Minoris* for the 90th Meridian West from Greenwich. Astronomical Time. Latitude 45°. Apparent δ and α with Greenwich Sidereal Time of Mean Noon, for the Year 1912.

Date.	Right ascension.	Declination.	Upper culmination.	Diff. for 1 day, U. C.	W. elongation.	Gr. sid. time of mean noon. Diff. 1 hour + 9.8565 s.
	h. m.		h. m. s.	m.	h. m.	h. m. s.
	1 27	88° 50′				
1912	s.					
Jan. 1	38.5	30.8″	6 46 25	3.95	12 40.8	18 39 07.5
15	24.0	22.1″	5 51 08	3.95	11 45.5	19 34 19.4
Feb. 1	06.0	32.0″	4 43 59	3.95	10 38.3	20 41 20.8
	h. m. 1 26 s.					
15	52 2	30.5″	3 48 43	3.95	9 43.1	21 36 32.6
Mar. 1	39.6	27.7″	2 49 32	3.94	8 43.9	22 35 40.9
15	31.2	24.1″	1 54 21	3.94	7 48.7	23 30 52.7
Apl. 1	25.5	19.1″	47 25	3.94	6 41.8	37 54.1
15	25.5	14.5″	23 48 26	3.93	5 46.7	1 33 05.9
May 1	30.9	9.9″	22 45 36	3.93	4 43.8	2 36 10.7
					E. elongation.	
15	38.9	6.5″	21 50 42	3.92	15 56.3	3 31 22.5
June 1	52.9	3.5″	20 44 05	3.92	14 49.7	4 38 24.0
	h. m. 1 27 s.					
15	06.4	2.0″	19 49 16	3.91	13 54.9	5 33 35.8
July 1	23.0	1.7″	18 46 38	3.91	12 52.2	6 36 40.8
15	38.2	2.5″	17 51 50	3.92	11 57.5	7 31 52.6
Aug. 1	55.9	4.7″	16 45 17	3.92	10 50.9	8 38 54.1
	h. m. 1 28 s.					
15	09.6	7.7″	15 50 28	3.92	9 56.1	9 34 05.9
Sep. 1	24.2	12.3″	14 43 53	3.92	8 49.5	10 41 07.3
15	33.8	16.9″	13 49 00	3.93	7 54.6	11 36 19.0
Oct. 1	41.7	22.7″	12 46 13	3.93	6 51.8	12 39 23.9
15	45.2	28.2″	11 51 14	3.93	5 56.8	13 34 35.7
Nov. 1	44.8	34.9″	10 44 23	3.93	4 50.0	14 41 37.1
					W. elongation.	
15	40.9	40.0″	9 49 16	3.94	15 43.6	15 36 48.9
Dec. 1	32.1	45.2″	8 46 13	3.94	14 40.6	16 39 53.8
15	21.6	48.9″	7 51 00	3.95	13 45.4	17 35 05.6
1913 Jan. 1	05.8	51.9″	6 43 53	3.95	12 38.3	18 42 07.1

The civil day begins 12 hours before the astronomical day; and the first period of the civil day answers to the last part of the preceding astronomical day—thus, May 15th, 15h, 56. 3m. astronomical time is the same as May 16th, 3h. 56. 3m. A. M., civil time.

To prevent a repetition of past errors in measurements and alignment, modern methods are replacing the crude devices of the past, and better instruments are taking the place of those formerly used. One of the greatest advances has been made in corner material. The old corner used throughout the plains country of the West, was a makeshift, which is now replaced by one that is practically indestructible, of cast iron, cement, and brass, with the township, range, and sections plainly indicated. What is of more material benefit, men of a different class are giving their attention to surveying in all its branches. The old-time local practitioner is giving way to the man who has a better education, and the attention of the Engineering Profession is being called to a vocation once much neglected.

DISCUSSION

Mr. Newbrough. W. NEWBROUGH, M. AM. SOC. C. E. (by letter).—To one who is familiar with Government surveying, as done under the direction of the General Land Office, on the public lands of the United States, the title of this paper is somewhat misleading, and is also defective in that it does not mention that the paper relates to the public system of surveys, as above mentioned. This, however, is a minor matter, and Mr. Sweitzer is to be thanked for the paper, as too little is available on this subject.

It is to be regretted that the author did not give fully the reasons for making the surveys he speaks of, and state for whom the work was done. This would be enlightening, as the owners of lands in the far Western States are usually farmers and ranchmen who could not afford to pay for such extended work as he describes, nor does the ordinary surveyor generally have a chance to make resurveys with such full equipment. The reason for this is that a man (or a company) owning a few sections or parts of sections, say from 320 to 3 200 acres in a township, generally does not wish (or cannot afford) to go to the expense of running 80 miles of lines, covering about 23 000 acres of land, in order to locate his holdings; of course, this is not so if the land is valuable, but, at the present time, such is not usually the case.

Mr. Sweitzer refers to the bad or careless work on the old surveys, and states that this was due to the contract system in vogue in former years. The writer, having been engaged on Government contracts in years gone by, and having officially and privately retraced thousands of miles of lines of this character, personally and with his assistants, is in general surprised at the good work that was done. It is true that there are surveys in Wyoming, Utah, Montana, and California, notably those executed from 1876 to 1884, when contracts were taken by the Benson, McCoy, Woods outfit, which are valueless or very bad, but these are exceptions.

Since 1890, when the system of examiners was started, the work has been good. Lately, however, the General Land Office has abolished the system of contracts, and does the work itself. This has been going on for about two years, and it will take some time to prove whether it is better than the contract system with examiners.

However, as the paper treats especially of the surveys where corners are missing, the writer will confine his discussion more closely to that subject.

Retracements, as defined in the "Manual of Surveying Instructions for the Survey of the Public Lands of the United States and Private Land Claims,"* mean the determination of the true bearings and dis-

* Obtainable from the Commissioner of the General Land Office, Washington, D. C., or from the Superintendent of Documents.

tances between the successive corners along the entire length of such a line; with details of the methods used. Mr. Newbrough.

The resurvey consists of a retracement, accompanied by the reconstruction of defective original corners and the establishment thereon of all the necessary new corners.* Consequently the resurvey includes the retracement.

On the subject of "Original Surveys," under which head Mr. Sweitzer quotes several Acts of Congress, it may be well to mention that the "Manual" gives the whole history of the establishment and maintenance of the United States system of public surveys, including all the Acts of Congress passed in relation thereto.

He also states that "any resurvey, in order to be legal, will have to relocate the original Government corners in their original positions." This is the case when a private surveyor makes the resurvey, but not when it is made by the Government, as is shown by the thousands of resurvey plats now on file in the various land offices. In many cases the new survey corners are a full mile from the old ones. The reason these are held to be legal is that the lands resurveyed still belonged to the United States, and the resurvey virtually is the original. Any patented lands are marked in their original positions by monuments and thus platted on the resurvey plats.

The above requirement is not absolutely insisted on by the Courts when it is manifestly unreasonable, as witness the case quoted on page 8 of the pamphlet issued by the General Land Office entitled, "Restoration of Lost or Obliterated Corners and Subdivision of Sections."

"In the case of an erroneous but existing closing corner which was set out of the true State boundary of Missouri and Kansas, it was held by the office that the surveyor subdividing the fractional section should preserve the boundary as a straight line."

The foregoing pamphlet is valuable, and should be obtained by everyone concerned in the work designated by its title. While Mr. Sweitzer's paper gives methods for handling large areas, this pamphlet will usually suffice to retrace or resurvey any small portion without rerunning the whole township. In addition to this, its rules are very practical. For instance, suppose the corner to Sections 14, 15, 22, and 23 is missing. Suppose the quarter corner to Sections 15 and 22 is in place, and the surveyor begins there and runs a random line east. At 1 mile he finds nothing, at 1½ miles nothing, at 2 miles nothing, and the same at 2½ miles, which brings him to the range line. Now, instead of running farther east, he must run north and south to try to find a range corner. The reasons for this are explained in the pamphlet. In the State of Wyoming this pamphlet has been incorporated bodily into the State Laws.

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*Manual for 1902, p. 79.

Mr. Newbrough. Mr. Sweitzer quotes several decisions, some of which relate to surveys other than those here treated of, but he makes no mention of what may be called the "law of acquiescence," or possession undisturbed for a number of years. In Wyoming, by law, this time is 10 years; in other States, other times are stipulated. This law gives the surveyor more trouble than nearly all the others, and this is caused by its enforcement by the Courts. Sometimes it is held to be good; at other times it is not. It seems to be elastic. Strange as it may seem, it is sometimes held good by the General Land Office. Witness the case of T. 24 N., R. 119 W., 6th P. M. This township, on the ground, by the original survey, had the following corners in it in the year 1904: SW. corner of Section 5 ($\frac{1}{4}$ corner to Sections 5 and 8), SE. corners of Sections 8, 17, and 20, and some others which do not relate to the case. On the ground, the sections of the north tier were $\frac{1}{4}$ mile too long, north and south. The plat in the land office showed the sections 1 mile long, or correct. Each 40-acre tract had on the ground an excess of acreage of about 40 acres. A road along the south boundary of Section 5 had been constructed soon after the survey was made, and extended clear across the section. If strict rules had been followed, this excess $\frac{1}{4}$ mile would have been distributed in the north and south tiers of sections lying east of the quarter corner to Sections 5 and 8. All the land in Sections 3, 4, and 5 had been patented and cultivated for years, and each settler who was fortunate enough to have this excess of 40 acres claimed it undisputed for years. In 1907 the Government made a resurvey of this township, and sent special examiners out in addition to the regular deputy. The result was that the road was presumed to be correct, according to the original survey, having been there for years, and each claimant got his 40 acres of excess for nothing. The writer should have mentioned that, while there were no other corners, there were several old fences which also helped this decision. There were no corners on the east boundary of the township, except the SE. corner.

In general, the law that makes the corners correct and unchangeable is a good one. When the ground is first surveyed, it is generally worth about $1.25 per acre. A man files a homestead or desert claim on such land; it is still worth no more, and if the closing on a section is within 50 links or so, not much acreage is taken from or granted to the settler, so that no harm is done. In time the land is improved—possibly sold to others—and becomes valuable. No harm has yet been done, because each man has known just what he was buying. It makes no difference whether the final owner has 155 or 160 acres. He has lost nothing.

The correction of errors by the Coast Survey causes no one any trouble, and so it is reasonable that they should be corrected.

This law practically applies to a lot in a town, as well as to the Government corners. The purchaser gets the lot where the original lot stake stood, whether this gives him 50, or 49.7, or 50.2 ft. Averaging, in resurveys, is only for the purpose of trying to equalize matters, and an original town stake will disarrange the most careful averaging, or even correct resurvey, every time. As Judge Cooley says: "It is not where the monument should have stood, but where it stands that governs." The method of correcting erroneous surveys on Government lands is by making a resurvey, and this is done quite frequently. One Act of Congress, five years ago, ordered the resurvey of about 400 or 500 townships in Western Wyoming, and the surveyors have been at this work ever since. Much of this land did not need resurveying, but that is another story. Mr. Newbrough.

As Mr. Sweitzer says, when pits have been dug in the grass-covered prairies of Kansas and Nebraska, they can be found, but when a surveyor makes a mound of earth in a clear, sandy country, and places four pits around it, in 12 months the pits are filled with sand and the corner is lost. This has been the case in Wyoming and Utah. Mr. Sweitzer's remarks on bearing trees are valuable.

The author should have stated that his "Township Partly Surveyed" could be fixed up if the SW. corner was in place or could be located, otherwise it could not. In his "Township Never Surveyed," he assumes that the exteriors are in place, or can be located. Usually, it has been the fate of the writer and his assistants to find some corners in place on the boundary of the township and some on the interior, but, with both combined, there were not enough to enable one to follow the author's method in full. Generally, the writer has had to run from interior corners to help locate exterior ones, going into the township in discussion and the adjoining ones, and then has frequently had to use much judgment in placing them. In such a case as this it would be well to study the pamphlet previously mentioned.

Exterior corners are supposed to be located in two directions only, as the township and range lines are the most important, and yet every missing corner should be replaced by checking from every possible direction. In doing this it will generally be necessary to run interior lines, and then it should not be necessary to re-run these lines when following Mr. Sweitzer's method. Care should be taken on this point.

The method given for an observation on the pole star at any hour (and taken at about sundown) is quick and accurate. Some years ago Mr. Baldwin, of the U. S. Geological Survey, suggested to the writer that when using this method, it would be well to make a scale on a narrow board, assuming a radius of about 300 ft. This gives about 0.1 ft. to the minute, which, of course, can be subdivided clearly enough to read 5″. The board is continually carried in the field, and is used as the mark when taking an observation. By making this board about 8 ft. long, it answers for the mark and for turning the azimuth of the

Mr. Newbrough. star. The writer has found that with care the observation can be taken within about 15″ on an instrument graduated to minutes. It is noted that Mr. Sweitzer takes his mark about 30 chains distant, but in his examples he reads his instrument only to minutes. It is hoped that, in his closing discussion, he will give the reason for this. One minute in 300 ft. is about 0.1 ft., and the writer sees no reason for taking a mark farther away, except possibly that it may be a natural object which may be of considerable size.

In Wyoming and Utah surveyors make great use of direct observations on the sun, and by taking four can generally come within about a minute. To one who is accustomed to these, the calculation is very rapid.

One of the most valuable features of this paper is the table for running east and west lines, which is a great improvement over the secant method.

Mr. LeBaron. J. Francis LeBaron, M. Am. Soc. C. E. (by letter.) This is an excellent paper, and should be in the hands of every civil engineer having field work to do in that part of the country covered by the United States system of Public Land Surveys.

Nearly every engineer who comes from the original Thirteen States, where this system is not in use, appears to be utterly ignorant of the fact that corners set by the U. S. Surveyors under this system are immovable and non-changeable, and when employed to re-survey a section line and re-set a lost section corner, they proceed to do the work without first obtaining copies of the original field notes, or the original township plat, without which they cannot make a legal resurvey. These notes and plats can be obtained only from the U. S. Surveyor-General for the district in which the land is situated, and not from the Register or Receiver. The latter will furnish a plat, but it is correct only as regards areas and has no courses and generally no distances marked on it. The plats from the Surveyor-General's office, on the contrary, contain all these data.

Too often the writer has seen surveyors re-run section and township lines without any plats at all, and without any copy of the original field notes, assuming every section line to be exactly 1 mile long and running true north and south, or east and west. Then, when the surveyor's most accurate steel tape measurements and transit lines failed to locate the corner on the fence or blazed line, he would establish a new corner and assure the land owner that his corner was absolutely correct, because so much care had been taken in the work. The writer has found corners of this kind set up in fields as much as 70, 80, or 100 ft. from the true position, and the whole neighborhood in a turmoil. It never seems to occur to these "surveyors" that they may be taking from or adding to the land of the adjoining owner.

It is customary for most cheap surveyors to establish corners in this way, never checking up from the opposite direction, because they say their clients object to the expense. In this they are countenanced by the majority of lawyers, who also show a surprising ignorance of the law of surveys. The writer has known a lawyer to counsel a client not to pay the surveyor's bill because he had gone to an opposite corner, a mile away, and run a check line south to fix the provisional corner which he had set by running from an established corner north. As the two corners came about 60 ft. apart, the true corner was set by proportional distances, and three of the four witness stumps were found. The trees had all been cut off, so that no blazes could be found at the corner or on any of the lines. The result threw the client's house on the wrong side of the line, and the lawyer, who had employed the writer, advised his client not to pay the bill, saying that the writer had done him no good, but rather harm, and had spent three-quarters of the time in surveying other men's land, alluding to the check lines, which it had been found necessary to run. Mr. LeBaron.

Lawyers, almost invariably, will insist that the survey shall be started from the first point named in the description in the deed, entirely oblivious of the Courts' rulings that this point is of no more dignity than any other in the description, and they generally presume to instruct the surveyor as to how the work should be done, somewhat as follows:

"I want you to start from the beginning corner, as given in the deed, run the exact course and distance, and set a stake there. That is all you have to do. It will not take you long. I suppose you make an allowance for the variation of the needle. The needle, you know, does not point exactly north, and you must make an allowance. This deed reads 'due north' so many chains. Now does that mean true north or the way the needle points? I suppose when you chain downhill you make an allowance, don't you, because I think the distance wouldn't come right if you didn't? I don't know how much you allow, but I suppose you have some custom about it. You see, this deed says so many chains and links, so you must measure it with a chain and links, and not any other kind of a measure, or I am inclined to think that the Court would reject your survey."

The writer has actually had these instructions given to him on several occasions, although the first sentence is directly at variance with all the decisions of the Courts, which are that marked lines and corners must govern, and not courses and distances. Every honest surveyor knows that courses and distances are only aids to help him find the true corner, and if the corner is lost it can only be re-established by running lines from all well-established corners and proportioning the errors. The legal mind, however, is peculiar.

The writer has read this paper with much interest, and heartily approves of it, but he is somewhat surprised that the author does

Mr. LeBaron. not mention the pamphlet published by the U. S. General Land Office, entitled, "Instruction for the Re-establishment of Lost or Obliterated Corners." This is official, and can be obtained free from the Commissioner of the General Land Office at Washington. In addition to the books mentioned by the author, "Dunn's Land Laws and Legal Decisions,"* will be found invaluable, and for several years *Engineering News* published a column of "Legal Decisions," which are very useful both to the surveyor and the engineer. Every chief of party on railroad location in the Public Land States should be provided with these books in order to compute correctly the land taken from private owners for right of way, and also to enable correct maps to be made.

As a general thing, railroad maps show the township and section lines running straight for scores of miles, and every section exactly 1 mile square, whereas, as a matter of fact, it is only in very exceptional cases that such lines are straight. On the contrary, almost every section line will vary in course and distance from every other, and the section just south of every township line will be found to fall short or exceed the normal length to a considerable degree, as the law of the Public Land Surveys provides that the surveys of the sections in a township shall commence at the southeast corner and proceed north and west. The township lines having been previously run and the section and half-section corners set on them, the subdivision of the interior sections proceeds until the north and west township lines are reached, when it invariably happens that the closing lines will fall short or over-run from a few links to one or several chains, and the law directs that this deficiency or excess shall be thrown into the last half of the section.

When the course of the last mile does not strike the corner of the section previously set on the township line, it is very often found that double corners have been established, one set for sections north or west of the township line and the other for sections south and east. Therefore, it is absolutely essential for the engineer to have copies of the original survey field notes, and run out each section through which the railroad passes, in order to compute the area of the fractional sections taken by the right of way, etc., as the distance measured on the ground between the section corners will seldom agree with the distance recorded in the field notes. The draftsman must have these notes in order to make correct maps.

As a general thing, however, all railroad maps are drawn with perfectly straight and regular section lines, ignoring all double corners and all excess or deficiency adjacent to the township lines. It is much easier for the draftsman, saves time, and looks so much better! Then people wonder why two maps never match by the section lines. The general method is to locate a section corner now and then, when

* Engineering News Publishing Company, New York.

the line runs near it, and, from these, lay off an exact checker-board of sections, without any reference to the field notes of the original survey or the official plats of the Surveyor-General's office. The result is confusion. Mr. Le Baron.

A. M. STRONG, ASSOC. M. AM. SOC. C. E. (by letter).—This paper calls attention to a subject which is of great importance in the growing Western States and of which the general public and many engineers seem to know very little. The rules governing retracement-resurveys, as outlined by the author, have been found necessary in order to hold to definite boundary lines after corners set on original surveys become destroyed. They should be followed on all such surveys, but many engineers and surveyors either do not know them or pay no attention to them. These rules apply not only to the retracement of section and township lines, but to the subdivision of the sections into smaller tracts, townsite surveys, and to many other classes of work. Mr. Strong.

Mr. Sweitzer touches on the principle of *pro rata* or proportional measurements, which many seem to find very hard to understand. On the old Government surveys a chain 66 ft. long was used on level ground and one 33 ft. long on rough ground. These chains were made up of 100 or 50 short lengths of wire connected by links. In the field these links became worn and stretched out of shape, thus lengthening the chain, or the wires became bent and the links kinked, thus shortening it. Each party was supposed to carry a new standard chain, and the old instructions read: "The chain in use will be compared and adjusted with this field standard each working day." In practice, one standard had to do for several parties, and adjustment was made only at long intervals. Even where the original chaining was done with reasonable care, as was the case under many of the contracts, a considerable difference will be found in retracing with a standard steel tape. The rule that "the length of such lines as returned * * * shall be held and considered as the true length thereof," simply means that the chain used on the original survey is to be considered the standard measure for that particular piece of work.

On minor Government surveys, such as mineral lands, and on right-of-way filings over Government lands, the instructions from the General Land Office read: "When a * * * line * * * intersects a section line, give course and bearing from the point of intersection to the corner of the public surveys at each end of the half mile of section line so intersected." It was long held that all such courses and bearings must be reported as given in the original field notes. In many cases this necessitated the filing of notes in which the measured distances were proportioned to fit the original notes. Later, this rule was changed to allow "the bearings and distances to be reported as found," thus acknowledging the differences which would be bound to occur except under work of the most expen-

Mr. Strong. sive class. A fruitful source of trouble is where rights of way on public lands have their location in sections computed from a few ties. Later, when the land is located and improved, it is found that the ditch, pole line, railway, or whatever it may be, is not occupying the land reserved for it by the Government.

In many districts only a few of the original corners can be found, and the common practice of starting from the nearest of these and laying off the land with true bearings and standard measurements results in as many sets of lines as there are corners from which to start. A given tract of land can usually be surveyed in less time in this way than by making a correct retracement-resurvey, and unless the owner of the land is well posted on correct methods, he will naturally go to the man who will do the work in the shortest time. As a result, many unnecessary disputes arise, for which the surveyor is mainly to blame. The permanent establishment of property lines so that fences and other improvements can stay where they are placed is usually worth more than a little land, and the extra time necessitated by the correct method is worth its cost.

Perhaps a few cases which have come to the writer's observation will illustrate some of these points. A certain township was surveyed in 1856, light wood stakes being used for the corners. It was partly settled in the late Sixties in large holdings. In late years these large holdings have been cut up, and a considerable increase in values has called attention to property boundaries. Recently, a number of retracement surveys have been made, and the only corners which can be found are in some rocky land along the northern and southern township lines. The surveys were made by different parties using one or another of these few corners as starting points and giving the land full measurement. It was soon found that in the center of the township there was an overlap of about 200 ft. between the surveys starting from the north boundary and those starting from the south. Practically, not one of the old fences or roads would fit either set of lines, and even the locations of the lots in the village were in dispute. A proper retracement survey, connecting all the existing corners and the oldest fences and tree rows, which were supposed to have been built or planted when more of the corners were in place, showed that by using a chain length of 65.34 ft. every point could be checked. The result is that no one is satisfied, and each property owner is inclined to believe that the survey which comes nearest to fitting his lines as he has them fenced, and still gives him his full area, is the correct one. The Courts will have to settle the disagreement between the different surveyors.

In another case in which the land was surveyed at about the same time, and settled under similar conditions, there now remains a township corner and three interior section corners. A number of recent resurveys have been made, some using one and some another of these corners as starting points, with the result that every property line in the district is unsettled. One of these surveys was made in connection with a very large water-development project, and right-of-way deeds were made out from it, describing the lands by lines joining points on the boundaries of legal subdivisions. A proper retracement survey, joining all existing corners and old land marks, showed that while the distances were about correct, the lines differed from the true meridian by nearly half a degree. The right-of-way deeds included valuable lands which it had never been the intention to convey. Mr. Strong.

In a desert valley surveyed in 1854 the corners were marked by oak stakes, charred on the points and set in earth mounds. It has been found within the last few years that there is water below the surface and the valley is being settled. The first settlers brought in a surveyor to give them the lines on the land. He located a few of the original corners, but as he was not able to make them fit exactly with the field notes, he claimed that they must be incorrect, and, starting from a standard parallel, made what he claimed to be a correct resurvey of the valley. The writer found that the length of the original chain must have been about 66.5 ft., and, working on this basis, has been able to find some remains of the majority of the original corners. In some places they are as much as 300 ft. from the corners as set for the first settlers.

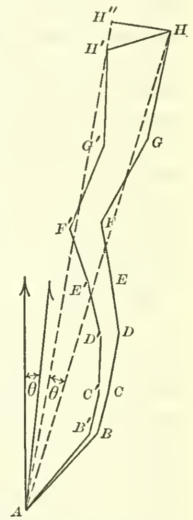
An engineer engaged in building a short branch railway had occasion to set the corner for a certain piece of property, and it differed so much from the accepted lines that the owner would not sign the right-of-way deed. When the county surveyor was called in, the railroad engineer explained that his work was tied in to a well-established corner on the township line 5 miles away, and he was positive that he had the correct point for this corner to within at least a couple of tenths. He probably still thinks that the county surveyor's statement, that this method of setting a corner might give a point as much as 200 or 300 ft. from the true one, showed that this surveyor was incompetent.

In each of these cases a little attention to the established rules in the first place would have saved much expense and trouble, to say nothing of the feelings of the engineers involved, and these are not exceptional cases by any means.

Leonard S. Smith, M. Am. Soc. C. E. (by letter).—The original land surveys of the United States Land Office are well-nigh completed, and the greater part of the public domain has passed into the hands of settlers; but, as these lands are divided up or are acquired by new owners, accurate resurveys become increasingly frequent and important. Mr. Smith.

Only those who have had to do with the relocating of the original monuments, and the distribution of the "surplus" and "deficiency," Mr. Smith. can adequately appreciate the difficulties involved in such retracements, or the need of uniting mature judgment with scientific methods. Mr. Sweitzer describes one of these resurveys in a most interesting way.

Another quite different type of resurvey, and one of very frequent occurrence in the older portions of the country, arises in the relocation of a meandering country road. Such a road, in broken topography, is very apt to be made up of many courses. The monuments and bearing trees referred to in the notes of the original survey, with but few exceptions, have been destroyed and lost long ago. The problem then is to retrace the line of the road as originally surveyed, the only guide being the original field notes and the two monuments widely separated by the various intervening angling courses. For example, in Fig. 6, A and H represent two existing original monuments, about 1.5 miles apart but connected by the courses shown. In such a problem the two chief sources of uncertainty relate to the actual meridian and the true length of the chain used by the original surveyor. Thus his meridian may have been a true, magnetic, or an assumed one; and his tape may have been longer or shorter than the standard length. Both these unknowns may be determined as follows: Using an assumed meridian, as near the original as can be determined, and beginning at one of the known monuments, run and mark on the ground a random traverse, using the bearings and distances taken from the original field notes, and continuing until the second known corner monument is reached. If, in the very rare case, the random line should happen to come out at this monument, it would show that the assumed meridian and the length of the chain used in the resurvey were the same as those used on the original survey. In such a case, such a random line would coincide with the true original line. Usually, however, the last corner of the random will fail to agree with the actual known corner. In this case, if the length and bearing of the closing line to such known existing corner be observed, the true positions of all the intermediate corners can be computed and marked by the following method:



The Points, A and H, are Original Monuments; All other Points lost.

FIG. 6.

Let $A\ B'\ C'\ D'\ E'\ F'\ G'\ H'$, Fig. 6,* represent the random line run on the assumed meridian, and $A\ B\ C\ D\ E\ F\ G\ H$, the original traverse, of which only A and H can now be found. The closing line, $H'\ H$, is observed to be N. 82° 32′ E. 63 ft. Connect A with H and H' and produce the latter until $A\ H'' = A\ H$. It is evident that if

* This example is taken from an actual survey by the writer.

the random line erred in direction only, then the point H' would lie in the arc, while if the discrepancy (H' H) were due to the chain only, then H' would lie in A H or A H produced. This shows that the position of H' with reference to the arc and the diagonal line, A H, will determine the kind of correction as well as the direction in which it is to be applied. Mr. Smith.

Thus $\frac{A\ H}{A\ H'}$ gives the length of the original chain in terms of the resurvey chain, while the distance, H H'', measures the twist or error in azimuth of the random line.

TABLE 3.—Random, Run on Assumed Meridian, Following Original Notes.

Course.	Bearing.	Distance, in feet.	Latitude.	Departure.		Total latitude.	Total de-parture.
			Northing.	Easting.	Westing.		
A B′	N. 39° 30′ E.	2 037.06	1 599.6	1 318.6	....	1 599.6	1 318.6
B′ C′	N. 24 15 E.	491.04	447.7	201.7	....	2 047.3	1 520.3
C′ D′	N. 9 45 E.	834.9	822.8	141.4	....	2 870.1	1 661.7
D′ E′	N. 4 0 W.	891.0	888.8	....	62.2	3 759.9	1 599.5
E′ F′	N. 9 15 W.	948.42	936.1	....	152.4	4 695.0	1 447.1
F′ G′	N. 31 20 E.	2 132.50	1 821.5	1 108.9	....	6 516.5	2 556.0
G′ H′	North	1 881.0	1 881.0	....	....	8 397.5	2 556.0

From Fig. 6 and Table 3 it will be seen that the bearing, H'' H', $= \tan.^{-1} \frac{2\,556}{8\,397.5} =$ N. 16° 56′ E., while the bearing of H' H is observed = N. 82° 32′ E. Hence the angle:

$$H''\,H'\,H = 65°\ 36'.$$

$$H\ H'' = 63 \text{ ft. sin. } 65°\ 36' = 57.4 \text{ ft.}$$

$$H'\,H'' = 63 \text{ ft. cos. } 65°\ 36' = 26.0 \text{ ft.}$$

$$A\ H = \sqrt{(\Sigma\text{Lat. of Random})^2 + (\Sigma\text{ Departure of Random})^2}$$

$$= \sqrt{(8\,397.5)^2 + (2\,556.0)^2} = 8\,777.9 \text{ ft.}$$

$$A\ H = A\ H' + H'\ H''.$$

$$= 8\,777.9 + 26.0 = 8\,803.9.$$

(1) The ratio of $\frac{\text{Original chain}}{\text{Resurvey tape}} = \frac{8\,803.9}{8\,777.9} = 1.00296$, that is, the original 100-ft. chain was 0.296 ft. too long (as compared with the resurvey tape as a standard).

(2) The error in the assumed meridian $= \tan.^{-1} \frac{57.4}{8\,803.9} = 0°\ 22\tfrac{1}{2}'$.

From Fig. 6 it will be seen that 22½′ is to be added to the random meridian, that is, it is to be swung clockwise.

To correct the corners set on the random line to their true position, the corrected line may be referred to the random line by adding

Mr. Smith. the 22½′ error in azimuth. This gives the new bearings in Table 4. Also, each of the original distances is multiplied by the ratio, 1.00296, thereby deriving the true distances in terms of the resurvey tape. These distances are also put in Table 4. Then the new latitude and departure may be computed, as well as the total latitude and departure of each corner on the true traverse.

The difference between the total latitude and departure of any corner on the random from Table 3, and the total latitude and total departure of the corresponding corner on the true line from Table 4, when applied to the corners on the random line, will give the field locations for the true corners, that is, the corrections to the random corners.

TABLE 4.—Co-ordinates of the Corners of the True Line.

Course.	Bearing.	Distance, in feet.	Latitude. Northing.	Departure. Easting.	Departure. Westing.	Total latitude.	Total departure.
A B........	N. 39° 52 1/2′ E..	2 079.2	1 595.7	1 333.0	....	1 595.7	1 333.0
B C........	N. 24 37 1/2 E...	492.5	447.7	205.2	....	2 043.4	1 538.2
C D........	N. 10 07 1/2 E ..	837.4	824.3	147.2	....	2 867.7	1 685.4
D E........	N. 3 37 1/2 W..	893.6	891.9	....	56.5	3 759.6	1 628.9
E F........	N. 8 52 1/2 W..	951.2	939.8	....	146.7	4 699.4	1 482.2
F G........	N. 31 42 1/2 E...	2 138.8	1 819.5	1 124.2	....	6 518.9	2 606.4
G H........	N. 0 22 1/2 E...	1 886.6	1 886.6	12.4	....	8 405.5	2 618.8

Table 5 has been prepared in a manner to show the corrections to each random corner. Thus corner D', when moved 23.7 ft. east and 2.4 ft. south of D' will give the original point, D, as located in the original survey.

TABLE 5.—The Corrections to the Random Corners for the True Corners.

Course.	Total, random survey.	Easting, original survey.	Correction to random corner.	Total, random survey.	Northing, original survey.	Correction to random survey.
B..................	1 318.6	1 333.0	E. 14.4	1 599.6	1 595.7	S. 3.9
C..................	1 520.3	1 538.2	E. 17.9	2 047.3	2 043.4	S. 3.9
D..................	1 661.7	1 685.4	E. 23.7	2 870.1	2 867.7	S. 2.4
E..................	1 599.5	1 628.9	E. 29.4	3 759.9	3 759.6	S. 0.3
F..................	1 447.1	1 482.2	E. 35.1	4 695.0	4 699.4	N. 4.4
G..................	2 556.0	2 606.4	E. 50.4	6 516.5	6 518.9	N. 2.4
H..................	2 556.0	2 618.8	E. 62.8	8 397.5	8 405.5	N. 8.0

The computations are taken from an actual survey made under the direction of the writer, and this survey has very recently been confirmed by the Wisconsin Supreme Court.

In the foregoing only the systematic or constant errors of the original survey are considered. As such ancient traverses were commonly made with an open-sight compass, some accidental error must necessarily be expected; for example, such errors as could come from local attraction of the needle and from reading the needle only to the nearest quarter degree. Under the conditions controlling such ancient surveys, it should not be expected that the relocations made by the foregoing method will exactly coincide with the original traverse points, but, if any traces of such original monuments or bearing trees still exist, they may most readily be found by basing sub-surface examinations for remains of any original stakes and bearing trees on the new locations of the traverse. Thus such investigations made by the writer at points on the "true" traverse here discussed disclosed undisputed and positive evidence of the stumps and roots of the original bearing trees, all within from 1 to 4 ft. of the computed positions. When such positive evidences of the original locations can be identified, the final traverse location, of course, must be made to agree with the recovered monuments, conforming to the legal maxim, that "courses and distances must yield to the identified original monuments." In such a case, the main use of the foregoing computations has been to point out on the ground the locus where detailed search for the original monuments or references may profitably be made. In case the investigations do not disclose the remains and position of the original monuments and bearing trees or pits, the writer submits that the relocation of the traverse as described herein is the most probable and certain of any which can be suggested. It is significant that the Courts have adopted this view. Mr. Smith.

Significance of Adverse Possession.—In making retracement land surveys, the surveyor frequently has to face a new complication, the significance of adverse possession in fixing land boundaries. Adverse possession has been defined as "an actual and visible appropriation of land commenced and continued under claim of right, inconsistent with and hostile to the claim of the true owner."* The surveyor who bases his retracement locations solely on the description of the land given in the deed, and without regard to long-continued adverse possession, is certain to lead his client into expensive and disastrous litigation.

While this principle of legalized "squatter sovereignty" is well established in the law of every State, it has not received the attention from surveyors which its importance would justify. A brief restatement of its most important and frequent applications should be of interest.

Rights claimed under "adverse possession" fall into two general classes: First, when the adverse user can show some written document which purports to give him legal title, but which, by reason of some vital defect, fails to do so, the user is said to hold adversely

* Texas Revised Statutes, Sec. 3 198.

Mr. Smith. "under color of title." Second, without such written document, his holding would be "without color of title."

In the first case, where the user has "color of title," the laws of most States require only 10 years of adverse possession of land to give perfect legal title; but, when a claim to land is not founded on some color of title, State statutes usually provide that 20 years of such adverse possession are necessary to perfect legal title.

The following classes of cases will be briefly discussed:

1.—Two abutting owners build a fence, accepting it as a division line, in ignorance of the fact that such fence is not on the true line described in their respective deeds, a section line, for example.

The Supreme Courts of all the States here hold that this constitutes adverse possession "without color of title," and after 20 years perfects title to the party thus holding adversely. (Burrell *v.* Burrell, 11 Mass., 294; Makepeace *v.* Bancroft, 12 Mass., 469; Darci *v.* Eulon, 116 Ill., 575; Wickwell *v.* Adams, 7 Con., 761; Gilchrist *v.* Magee, 9 Yerg., 455.)

It should be noted that in this case the boundary may be established by monuments as well as by "visible boundary." Obviously, no resurvey can change or fix boundaries against adverse possession of this kind, after the statute of limitations has run.

2.—If such division fence had been built, not as a boundary, but as a matter of convenience or necessity, recognizing distinctly that it was not the true boundary, the Court decisions are practically unanimous, but to the opposite effect—the fence, however long the lapse of time, will not become the legal boundary. A survey is here necessary to fix the boundary. (Fairfield *v.* Barrette, 73 Wis., 463; Burnell *v.* Maloney, 39 Vt., 583; Griscom *v.* Murphy, 110 Ill., 274; Bunce *v.* Bidwell, 43 Mich., 542; Cole *v.* Parker, 70 No., 372.)

3.—The third class includes the greater number of cases which actually occur, namely, where the abutting owners build the fence supposing it to be on the true line, but under no distinct agreement to abide by it as the line—the case of tacit consent. On this case, unfortunately, the agreement of the Courts is not unanimous. The greater number of Supreme Courts, however, have held that this belongs in the same class as the first case discussed. The Supreme Courts of Maine and Missouri, however, followed at times by those of a few other States, have held that where an error has been made in marking and holding to a property line through ignorance and mistake, and with no intent to go beyond the true line, the doctrine of adverse possession does not apply. As a result, in such States, the true line when discovered holds, and the erroneous fence line must yield.* The

* Because of a growing distrust of the Courts, and the excessive cost of litigation, boundary lines are frequently established by mutual agreement. If such agreement be permanently monumented and duly recorded, this method of fixing the line is most satisfactory to all parties concerned. It hardly need be said, in such cases, that the line so agreed to becomes at once and for all time the true boundary.

following recent decisions of the Wisconsin Supreme Court support the former view: (Bishop *v.* Bleyer, 105 Wis., 330; Wollman *v.* Ruehle, 104 Wis., 693; Welton *v.* Poynton, 94 Wis., 406; Meyer *v.* Hope, 101 Wis., 123; Gilman *v.* Brown, 91 N. W., 227.) Mr. Smith.

The writer will now consider more fully the difference, as to ownership of a disputed strip, between those cases where the adverse possession is "with color of title" and those in which it is "without color of title."

When, in the case of two adjoining pieces of property, the calls of the two deeds, with reference to the common boundary, are identical, and the fence line or other visible boundary conflicts with the identical description, then adverse possession of the disputed strip must be "without color of title." Both deeds call for the same line; neither sanctions any departure from that line; hence the adverse possession can have no color of title, and, to be effective, must run 20 years; but, when the calls of the two deeds conflict in regard to the common boundary, in such a way that the two properties, as described, overlap, there is adverse possession "with color of title" to the extent of the overlap. In this case, the adverse possession will give perfect title in 10 years.

The foregoing gives briefly the essence of the doctrine of adverse possession as between individuals. It now remains to consider the effect of adverse possession against the public. Here the doctrine of the Common Law, *"Nullum tempus occurrit regi"* (time does not run against the king), has long been accepted as the rule of law, the place of the king being taken by the State or municipality. Court decisions are practically unanimous in declaring that no occupancy of a street or highway, however long continued, will give the trespasser legal title to the public land thus held. The same holds as regards any real estate owned by the public—a school ground, public park, court house, or jail lot—also of State or United States lands, forest reserves, etc.

For cases supporting this view, see Charlotte *v.* Pembroke Iron Works, 82 Me., 391; O'Conner *v.* Pittsburg, 18 Pa. St., 187; Arundell *v.* McCulloch, 10 Mass., 70; Milham *v.* Sharp, 27 N. Y., 611.

For contrary decisions see Knight *v.* Heaton, 22 Vt., 480; Beardslee *v.* French, 48 Am. Dec., 507; Weher *v.* Chapman, 42 N. H., 326.

Regarding corporations not public, the doctrine is the same as toward individuals. In the case of railroads, however, Court decisions are inconsistent and conflicting. The growing tendency to regard a railway as a public corporation tends to give its right of way more and more the character of a public highway, against which adverse possession cannot run; while, in the older view of it, as a private corporation, its lands, like those of other private corporations, are as subject to adverse possession as those of an individual.

Mr. Smith. In the following cases, the Courts held that adverse possession ran against the railroad corporation: Illinois Central *v.* O'Connor, 154 Ill., 550; Pittsburg, etc., Ry. Co. *v.* Strickley, 58 N. E., 192; Pollock *v.* Mayville, etc., Ry. Co., 44 S. W., 359; Spottiswoode *v.* Morris, etc., R. Co., 61 N. J. L., 322; Babbitt *v.* Southeastern R. Co., 9 Q. B. D., 424; Ry. Co. *v.* Hauken, 140 Iowa, 372.

For *contra* cases, see Southern Pac. Co. *v.* Hyatt, 64 Poc., 272; Powell *v.* R. Co., 215 Mo., 339; McLucuc *v.* R. Co., 67 Neb., 603; R. Co. *v.* Baker, 183 Mo., 312; R. Co. *v.* Ely, 25 Wash., 384; R. Co. *v.* Townsend, 190 U. S., 267.

The total aggregate of property lines which have become fixed by adverse possession is very great and is constantly increasing. The surveyor who has occasion to make relocations of old lines should know the general provisions of the law on this subject, both in his own and his client's interest. By this it is not meant that the surveyor should usurp the place of judge and jury. His duty as an expert is only to ascertain and report the facts to his clients. A knowledge of the significance of adverse possession, however, will in many cases greatly assist him in sifting out and interpreting more or less discordant facts. The surveyor should never lose sight of the fact that the Court and jury have the final decision as to the proper relocation of the line. They, at least, are not likely to overlook the authority which the law gives to long-continued possession.

Mr. Hinckley. H. V. Hinckley, M. Am. Soc. C. E. (by letter).—Two points occur to the writer which may be of interest:

First.—On an east and west section line in the interior of a township in Oklahoma, a section corner and a quarter-section corner, ½ mile apart, were missing. There were no witness trees. The county surveyor reset the corners. Property owners on the north of the line took the matter into court and, by "preponderance of testimony," showed that the original corners had been 24 and 48 ft. farther south than the county surveyor indicated. The surveyor's work was declared void in spite of the fact that the two corners (as located by evidence) were off line to the extent of the figures given.

Second.—The Dawes Commission, in allotting tribal lands in Indian Territory, cut some of the lands into quarter quarters, using stone posts for the "$\frac{1}{16}$" corners. Townsite surveyors later subdivided some of these same lands, using oak stakes for corners. The Interior Department instructed that the (permanent) stone corners set by the Dawes Commission were not official and that the (temporary) stakes set by the townsite surveyors (who had been commissioned as such) were official. Until tested in Court, this is a case where a stake beats a stone. As the discrepancies seldom reached 10 ft., the matter may never be tested.

Mr. Davis.

James L. Davis, Assoc. M. Am. Soc. C. E.—The author does well to bring out clearly the ease and simplicity of referring surveys to the true meridian. The textbooks, for the past ten years or more, have described various simple and easy methods for doing this work, yet it is not very commonly used.

There are three or four very simple methods: Observations on Polaris and on the sun serve for about all the cases which are met. The simple methods of observing Polaris are those at elongations, east or west, and at culmination. Observations may also be taken at any time Polaris can be seen, and the reductions may be made by using the hour-angle and azimuth tables published by the United States Land Office. The sun can be used with fair results, taking both forenoon and afternoon observations. All these methods can be used with comparative ease and rapidity, and give meridians correct within about 1 or 2′, with ordinary transits in the hands of observers of no special training. All that is required is a little practice on the part of any one who is a good manipulator of the transit and has sufficient mathematical knowledge to reduce trigonometric formulas.

The method of referring to the true meridian is especially valuable for surveys for railroads, canals, highways, or any survey requiring a long traverse and in which there is no opportunity for checking by a closure.

In making a traverse survey in October, the speaker has, at the end of the day, for several days in succession, the time of quitting work being within a few minutes of the time of the elongation of Polaris, when set up on the last station, taken direct and reverse observations, requiring not more than about 15 min. In this manner each day's work was checked, and any necessary corrections for azimuth were made by distributing the small error back through the work of the day.

The method is also valuable in land or town surveys, where the object is to determine areas or retrace old boundaries, and has the great advantage, whether applied to traverse or other surveys, that, as long as the record is preserved, it is only necessary to recover one point, or one monument of the survey, in order to re-establish the entire survey. Usual methods, working from the co-ordinate system, require two points to be preserved, while only one is necessary by this method in order to re-establish the survey within the next 100 or 200 years, or whatever length of time the record of the survey is preserved.

By taking the mean of several direct and reverse observations of Polaris at elongation, results correct within 10 or 15″ may be obtained.

The method used by Mr. Sweitzer in checking his determination of azimuth from equal altitudes of the sun seems worthy of more emphasis, as it is an independent process and simpler than the method

Mr. Davis. described, though an error in manipulation or computation would not be so readily revealed. In brief, it consists of bisecting the horizontal angle between the sun's forenoon and afternoon positions at equal altitudes, with proper correction for the sun's change in declination. As an independent method, an object is selected as an azimuth mark, and the horizontal angles are measured between this mark and the sun's forenoon and afternoon positions. The mark, preferably, should be to the left of the sun in the morning observation, then the mean of the two vernier readings gives the bisector of the angle. The following formula, a modification of the one used by Mr. Sweitzer, gives the correction to the mean of the two vernier readings: $\frac{D}{2 \cos. \phi \sin. t}$, in which D is the total change in the sun's declination between forenoon and afternoon observations, in seconds, ϕ is the observer's latitude, and t is the hour angle or one-half the time between observations, expressed in degrees (very nearly).

TABLE 6.—Hourly Motion of the Sun in Declination.

Day of Month.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1............	+12″	43″	57″	58″	45″	21″	10″	38″	54″	58″	48″	23″
5............	17	46	58	56	43	17	14	41	56	58	46	19
10............	22	49	59	54	39	12	19	44	·57	57	42	14
15............	28	52	59	52	36	7	24	47	58	56	38	8
20............	32	54	59	49	31	+ 2	28	49	58	54	34	− 2
25............	37	56	59	47	27	− 4	32	52	59	52	30	+ 4
30............	41	..	58	46	23	9	36	54	59	49	25	10

For an observer in north latitude, the correction is subtracted from the mean of the vernier readings if the sun is going north, and added if it is going south, the circle reading from 0° to 360° in clockwise direction. This method has the following advantages: The use of the Ephemeris is not required, only a small table of hourly changes in the sun's declination at 5-day intervals (Table 6), which, with the foregoing formula and the simple rules for its application, may be copied in the field notebook; it requires no correction of the observations for refraction or semi-diameter; it is not necessary to solve a spherical triangle; and it is possible to make the observations with a transit not equipped with a vertical arc, provided the telescope may be clamped in altitude. The speaker obtained this method from the catalogue of an instrument maker, and gives it here believing it is worthy of wider publication. He has never made use of this method, on account of the time required between observations. It is practicable, however, with a transit equipped with a vertical arc, to note the sun's altitude in the morning observation, remove the instrument and spend the interval between observations at other work, and return to the station for the afternoon observation.* Mr. Davis.

A. T. Parsons, Assoc. M. Am. Soc. C. E. (by letter).—The writer has read with interest Mr. Sweitzer's paper and the comments thereon, particularly those of Mr. LeBaron. Mr. LeBaron's experiences with lawyers, and it might also be said with owners, seem to have been similar to those of the writer. Too much stress cannot be laid on the importance of finding out where the original corners were, and where they should have been, and that the former should rule against the latter is a fact of which owners, lawyers, and "near" surveyors are frequently oblivious. Mr. Parsons.

The necessity of having the complete field notes as well as the original plat of the township is not appreciated by many surveyors. Where a survey in a mountainous country has been carefully made, and copious topographical notes have been taken, it is frequently possible to restore lost corners, even when lumbering operations and forest fires have destroyed many of the bearing trees; but mounds, pits, and stakes soon disappear on the open, sage brush plains, and it is here that the Land Office pamphlet, "Instructions for the Re-establishment of Lost or Obliterated Corners," is valuable. It is surprising that Mr. Sweitzer does not mention this pamphlet. If a surveyor follows the instructions given, he may be reasonably sure that the Courts will sustain him.

There is one rule in the instructions that the writer would like to see amended, namely, that for the re-establishment of lost interior section corners. This rule says in effect that they must be placed at the intersection of lines drawn east and west and north and south from the nearest recoverable corners on each side. When one considers that the "Manual" directs that, in ordinary cases, the original interior section corners should be set by running lines north from the corners already set on the south line of the township, it would seem that proportioning the distance between the nearest corners on the north and south would give a result as good as, or better than, the method prescribed, and it would frequently save a great deal of work. This is proposed, of course, in case the field notes show that the usual method was followed on the original subdivision.

There is one way in which a surveyor, with the best of intentions, may cause a lot of trouble, and this has not yet been mentioned in the discussion. Suppose a man has 40 or 160 acres adjacent to an established corner of some kind. He takes a man out there in a hurry, is unwilling that sufficient time should be taken to set the

* Since the foregoing was prepared, it has been learned that the method of determining the meridian from observations on the sun at equal altitudes is described, in substantially the same form, in Breed and Hosmer's "Principles and Practice of Surveying."

Mr. Parsons. corners correctly, and the surveyor is persuaded to lay off a square piece of land of the required dimensions.

At the time, the owner is duly impressed with the fact that the stakes are only approximate, but time passes, a dispute arises with a neighbor, and another surveyor is called in and sets the corners correctly and somewhat at variance with the other stakes. The owner then declares, "Why I paid——— $10 to set those corners, and he did the work wrong," and proceeds to damn all surveyors impartially.

Mr. Carpenter. J. C. Carpenter, Jun. Am. Soc. C. E. (by letter).—This paper deals with a subject which the writer has never seen treated before. Apparently, discussions of this sort are out of harmony with the present idea of engineering, and the supposition is that they would receive no welcome where works of tunneling, bridge building, and relatively exact subjects hold full sway without a suggestion of the "new element of error."

Most of the quotations given by Mr. Sweitzer are condensed in the pamphlet, "Manual for the Restoration of Lost and Obliterated Corners," issued by the General Land Office. The surveyor who works on resurveys in the Western States will find it to his advantage to have the rules of that book at his tongue's end, especially if called into Court. The three principal points emphasized in that pamphlet are as follows:

If the original corner, or any satisfactory evidence, such as bearing trees, witness marks, etc., which would determine its exact position, can be found, this corner should be perpetuated, whether or not the location corresponds with the field notes, unless there is a question as to the genuineness of the marks in the field.

The surveyor, in re-establishing a lost or obliterated corner, should consider the method of procedure taken by the original surveyor, and attempt to retrace the lines in the same manner.

The method of locating the corner, if all evidences in the immediate vicinity are destroyed, is, in general, by survey from the nearest known corners. Mr. Sweitzer makes little mention of the use of proportional measurement in the relocation of a corner. For relocation of corners common to four sections, as well as those common to four townships, this method is used. Measurements must be made north and south, and east and west, to the nearest existing corners, and the corner must be established by proportioning the new measurements to those given in the field notes of the original survey. This does not apply to corners on township or range lines which are not common to four townships, for, as they were originally established in running the township or range line before any subdivision was commenced, they must be relocated in the same manner, by measurements and line to the nearest existing corners on that township or range line, as the case

Mr. Carpenter.

may be. Hence it is evident that if, in relocating a lost interior section corner, no reliable corners are found before the township line is reached, and no corner is found there, measurement at right angles to the township line should cease and the township line corner should be re-established by survey along that line, after which the measurement should be made to this new corner and the interior corner should be established by proportional measurement. In relocating a corner on a township or range line (not common to four townships), measurements must be made to corners in the interior of the adjoining townships to check its location. However, if the location does not "check," there is no course outlined for the surveyor.

Quarter-section corners are located midway between section corners except on the north and west tiers of sections, where the over-plus or shortage is all thrown into the north and west quarters, respectively. These north and west tiers cause more trouble than any others. It is evident, in many cases, that the surveyors who subdivided the townships did not run to the township line in the last tier of sections, but simply ran out 40 chains on a computed course and estimated the remaining distance for their field notes. In the resurvey of townships in rolling country, where the surveyor was careless, the writer has found these corners out of line as much as 3 chains, and the distance was found to be at fault in the same proportion. As a general rule, if the quarter-corner common to Sections 30 and 31 is found to be north of a true line between the section corner and the corner on the range line, the quarter-corner to Sections 19 and 30 and those north of it will be found north of the true line also. In relocating these closing corners, it is well to be absolutely sure that the original corner cannot be found, before the line is straightened and the proportional measurement rule is applied.

In resurveying a township, the surveyor should attempt to put himself in the same mood as that of the original surveyor until the corners are located, and then slip out of that mood and tie in the corners so that he and his successors can find their location without adopting any method but that of a careful survey. A surveyor working in the same territory for a considerable length of time will become familiar with the methods adopted by the original surveyor, and be able to judge what moves his predecessor made. The writer has found it advisable to follow the footsteps of the original surveyor as closely as possible. The township and range lines are run first, although it is not advisable to run the whole way around the township at first. Enough of the township and range lines should be run to provide work for two or three days in the interior, the line being "tied up" on an original corner when left. The surveyor then starts at the south corner to Sections 35 and 36 and follows the original surveyor's steps through the township. If possible, the corners are set as the survey proceeds, Mr. Carpenter. but, if there are any lost corners in dispute, it is well to leave them for two or three days and talk the matter over with people who are familiar with the local situation. The writer has found it to his advantage to cultivate the acquaintance of the old settlers in the township, unless their veracity is questionable, and from talking with them and from local opinion has been able to find many corners which were supposed to be lost. If, after returning to the corner, the surveyor is unable to find the original marks, he must follow the rules for the relocation of obliterated corners, and should have, on his return, enough data to locate the corner satisfactorily.

A word should be said as to the value of improvements such as groves of trees, permanent fences, and even head lines of fields, which were made when the corner was plain, in deciding the location of a lost corner. A location arrived at by measurements from a row of trees set out when the corner was plain, such location being supported by two or more reliable affidavits, is undoubtedly infinitely better than one arrived at by measurement from corners half a mile or more away.

Before leaving the township, the surveyor should satisfy himself that there are no objections to his corners as set. If he finds objections he should make thorough investigations to be absolutely sure that his location is correct and that no signs of the original corner can be found at any other point. Should he find that his location does not agree with the original survey, and that the original corner may be located, he should lose no time in changing his corner and explaining to the local land owners his reasons for locating his first corner as he did. A little time spent in explaining these minor points will save endless bother later by establishing confidence in the remainder of the survey.

Mr. Sweitzer's description of the identification of an original corner by the examination of pits and of the soil is interesting and realistic. By this method the writer has identified corners which were forty years old and had been covered with a road grade for ten years or more. In Eastern Dakota the practice was to set a charred stake, post, mound, and pits. There is a rumor current that all the original surveyors smoked, that they lighted their pipes at each corner, and stuck the charred match in the ground for a stake. Experience sometimes seems to prove this rumor true.

As to the carelessness of the first surveyors, the writer offers as proof three instances from his experience:

In Estelline Township, South Dakota, the line one mile west of the east boundary makes an angle at each section corner, first to the right and then to the left, of approximately 5°, but the notes show not more than 10′ variation at any corner. The line is paralleled by a telephone line and may be plainly seen, zigzagging across the valley, from a hill just north of the Village of Estelline.

The town liner who surveyed Flandreau Township and the vicinity, located the northeast corner of that township about 20 chains north and 20 chains east of the point where it should be. The error was not corrected, but was reported by the man who subdivided the township and is a part of the field notes. Mr. Carpenter.

A lake on the boundary line between Clark and Codington Counties is actually half a mile south of its position as shown on the field notes. The "subdivider" did not report this error, but continued it, so that both the range line notes and the subdivision notes show it to be half a mile north of where it actually is.

N. B. Sweitzer, Assoc. M. Am. Soc. C. E. (by letter).—Mr. Newbrough brings up some interesting questions. Having been a deputy surveyor, he has evidently handled his questions at first-hand, and has seen the result of poor and inaccurate surveys. It has evidently been his good fortune to be in a section of the country where more than the average number of old Government surveys were in good condition. Unfortunately, much of this work was not only poorly executed, but was fraudulent in the extreme. However, this paper and the discussion are for the purpose of rectifying these old evils, and not for condemnation. Mr. Sweitzer.

As stated in the paper, the original surveys were made under authority of Congress, supplemented by such rules and regulations as the Executive Department considered pertinent to field procedure. The deputy surveyors, comprising a class of men who looked on all surveys from one angle, executed this work in the field, directly under the supervision of the surveyors general of the various States; and it is harder for men of this class to adjust the resurveys than for civil engineers or others who have never had to deal with original surveys, because the former are blinded by the apparently arbitrary authority held by them while doing original work. It might be well to state that, after a survey has been legally accepted, especially when titles have passed, no surveyor has the right to disregard the original Government monuments. In nearly all the States west of the Mississippi a glance will reveal the conditions at present caused by ignorance of this question, and the attempt to replace arbitrarily the original surveys with others having areas which do not correspond with, and having corners which are not in the locus of, the original, which result in giving conflicting deeds to different pieces of land.

Mr. J. Francis LeBaron very happily reiterates that question when he gives examples of this same condition. Mr. Strong also recognizes this fact.

In regard to the difference between a "resurvey," as set forth in the "Manual of Surveying Instructions," and a "retracement-resurvey," as used in this paper, attention is called to the following definitions:

Mr. Sweitzer. "A lost corner is one whose position cannot be determined, beyond a reasonable doubt, either from original marks or reliable external evidence." (Pamphlet.)

An obliterated corner is one of which no physical evidence remains of its establishment. Its location may be identified by "one who knows" and remembers the true locus of the corner, as built by the original surveyor.

A proportioned corner is one which has been re-established in its proper position, according to the original field notes, when no physical evidence of the original corner remains and its former location cannot be positively determined by the testimony of "one who knows."

"The retracement of a township boundary, or other lines of survey, consists in the determination of the true bearings and distances between the successive corners along the entire length of such a line; and the data thus obtained will be embodied in the field notes together with detailed particulars of the methods employed." ("Manual of Surveying Instructions.")

"The resurvey of a township boundary or other line of survey consists of a retracement of such a line accompanied by the reconstruction of defective original corners and the establishment thereon of all the necessary new corners; and the detailed particulars of the entire operations will be embodied in the field notes." ("Manual of Surveying Instructions.")

A retracement-resurvey of a former survey is the determination of the true bearings and distances between, and the restoration of, successive original corners, the locations of which have been ascertained, either by physical evidence, the testimony of "one who knows," when "obliterated," or by proportional measurement, based on the original plat, when "lost," the detailed particulars of the entire operations being embodied in the field notes.

It is apparent, therefore, that a "resurvey," as defined by the Manual, and a "retracement-resurvey," as set forth in this paper, are entirely different. Further, in the definition of a resurvey, the question of actually following and perpetuating the original corners is left somewhat in doubt by the latter part, which says, "and the establishment thereon of all necessary new corners." As a matter of fact, numerous official resurveys have been made in which the original survey has been retraced and resurveyed, as far as alignment is concerned, and new corners placed at distances other than those indicated by the original plats, making the miles and half miles 80 and 40 chains, and placing new and independent corners at these distances. This attempt at making a retracement-resurvey has evidently been confused by the foregoing quoted definitions. In retracement-resurveys, the law requires that all new corners be placed in the locus of those on the original survey. This is as it should be, for the corners mark the boundaries of lands deeded and to be deeded. The fact that the old Government deputies refused to recognize the authority of the Courts, and the further fact that they failed to interpret correctly the definition of a "retracement," are responsible for resurveys being executed in this manner. Mr. Sweitzer.

Along this same line, Mr. Newbrough states:

"In the case of an erroneous but existing closing corner which was set out of the true State boundary of Missouri and Kansas, it was held by the office that the surveyor subdividing the fractional section should preserve the boundary as a straight line."

This is undoubtedly a proper view to take; but the writer has yet to find where this case has been taken to the Courts. As previously stated, this view is "held by the office." It would be well to differentiate between suggestions made in this way and the final decisions made by Courts, as embarrassing situations might be avoided.

Mr. Newbrough also states:

"This is the case when a private surveyor makes the resurveys, but not when made by the Government, as is shown by the thousands of resurvey plats now on file in the various land offices. In many cases, the new survey corners are a full mile from the old ones. The reason these are held to be legal is that the lands resurveyed still belonged to the United States, and the resurvey virtually is the original."

As a matter of fact, the Courts have not held that this method of resurvey is legal; on the contrary, they have held it to be illegal. Again, it should be remembered that Sections 16 and 36 are school lands, and are deeded to the State immediately after the original survey is accepted, the State automatically securing its title thereby. The boundaries of the lands secured by the State, as well as by individuals, are indicated by the original corners on the ground, as shown by the field notes and plats of the original survey; and if new corners are set a mile from the original, then they are just a mile from where the Courts hold they should be, and do not indicate the boundaries of the land described in the original deeds from the Government.

The pamphlet, "Restoration of Lost or Obliterated Corners," was written, as indicated therein, for "county and local surveyors"; and its purpose was to aid in relocating, one corner at a time, the boundaries of small areas. The pamphlet is a valuable one, and every engineer should have it. However, the methods suggested therein require more field work and are more costly than those outlined in this paper. Furthermore, the suggestions made by the writer are as adaptable for small areas as for large ones. It will be noted, in that pamphlet, that retracements should be made to adjacent corners, to prove the location of the corners sought. Therefore, there is more work in the location of a single corner, by that pamphlet, than indicated by this paper—in fact, the retracement-resurvey method is shorter.

Mr. Newbrough gives the following example:

"In addition to this, its rules are very practical. For instance, suppose the corner to Sections 14, 15, 22, and 23 is missing. Suppose

Mr. Sweitzer. the quarter corner to Sections 15 and 22 is in place, and the surveyor begins there and runs a random line east. At 1 mile he finds nothing, at 1½ miles nothing, at 2 miles nothing, and the same at 2½ miles, which brings him to the range line. Now, instead of running farther east, he must run north and south to try to find a range corner. The reasons for this are explained in the pamphlet."

Mr. Newbrough's explanation is indefinite, in that he recites a number of missing corners, but does not state how they should be replaced, whether on a straight line or by proportional measurements. No one doubts that the range line limits the proportion in the township. Taking the latter example: Before subdividing a township, the exteriors thereof should be determined from the original corners, or (where they cannot be found), by proportional measurement, based on the original plat, or by competent evidence. Now, were the methods suggested in the "Lost or Obliterated Corners" pamphlet followed, the section corners would have to be placed in their original position by a double system of rectangular co-ordinates for each section corner from lines run north, south, east, and west to the nearest known corner. Then new lines would have to be run and a new survey made to relocate the quarter corners. Thus, by using that pamphlet, it will be seen that two surveys are necessary, one to locate the original section corners and one to locate the quarter- or half-mile corners, as these have to be on a straight line, midway between the section corners, except on the north and west tiers of sections, where a similar process would have to be used. In other words, the method of relocating corners by the "Lost and Obliterated Corners" pamphlet has to be done entirely on the ground, while the method suggested in the paper eliminates a great part of the mechanical work and substitutes that of calculation, which is more accurate than random lines measured on the ground, but arrives at the same result in a shorter way. It can be readily seen that, to put in a line of missing corners, say from the corner of Sections 33 and 34, on the south boundary of a township, to the corner of Sections 3 and 4, on the north boundary, by using rectangular co-ordinates instead of calculated proportional course and distance, as the writer has suggested, the field method would be much more laborious, and hence less accurate.

Concerning the position taken by Mr. Newbrough in regard to the "Township Partly Surveyed," in which the writer assumed the exteriors to be in place, he states:

"Usually, it has been the fate of the writer and his assistants to find some corners in place on the boundary of the township and some on the interior, but, with both combined, there were not enough to enable one to follow the author's method in full."

The writer is inclined to think that Mr. Newbrough has not read this paper carefully, and that he has not read the pamphlet on "Lost or Obliterated Corners" as fully as he should. It is expressly stated in the paper that, before commencing the subdivision of a township for a retracement-resurvey, the town and range lines should be relocated; and in the example stated it was assumed that such was the case, the corners which could not be found having been relocated by proportional measurement. They may be located by corners near or distant, as long as they are determined by the nearest known original corners and by the proportional measurement based on the original plats. No one will doubt that this can be done. Mr. Sweitzer.

In regard to the statute of limitations, Mr. Newbrough states that the writer does not mention what may be called the "law of acquiescence," or possession undisturbed for a number of years. Attention is called to this under the heading, "Resurveys," immediately after mentioning Justice Cooley, of the Supreme Court of Michigan, as follows:

"Keeping the justice and equity of the foregoing always in mind, it is also well to remember that adverse possession does not run against the Government."

Every engineer or surveyor who undertakes to make retracement-resurveys should have at least a fundamental knowledge of the law applicable to the technical work of running the lines; and the writer took it for granted that this phase of the subject was known to every one attempting to adjust property lines. As Mr. Newbrough states, this law seems to be elastic; and, after reading the various decisions, the writer has come to the conclusion that every case should be tried on its merits. No iron-clad rule can be laid down. It would probably be a good thing, however, for surveyors not to get into the judiciary, but to warn the land holders involved, when such questions come up, and, if they see fit, call in an expert on this work, such as a good lawyer. The main thing to which the writer wishes to call attention is that the statute of limitations does not run against the Government. As suggested by Mr. Smith, "time does not run against the king (*Nullum tempus occurrit regi*)." It would be well to remember that the township is like a checker-board, especially when this question is involved; and the very fact that the Government or State owns land within such township makes it hard for the individual, except in extraordinary cases, to plead successfully the statute of limitations.

In regard to the finding of corners which are some forty odd years old, Mr. Newbrough states:

"When pits have been dug in the grass-covered prairies of Kansas and Nebraska, they can be found, but when a surveyor makes a mound of earth in a clear, sandy country, and places four pits around it, in 12 months the pits are filled with sand and the corner is lost."

Evidently, he does not comprehend the writer's idea, which is that such a corner is not lost, whether in Kansas, Nebraska, or any other State. The fact that Nature fills these pits with material foreign to the surrounding soil simply perpetuates them; and the method by

Mr. Sweitzer. which they are found, suggested in the paper, is the key-note for locating properly the original corners in the prairie States. Mr. Newbrough should re-read that part of the paper.

Instead of the sky-line mark which the writer uses for daylight observations for azimuth, Mr. Newbrough suggests a board 8 ft. long, using a scale, assuming a radius of about 300 ft., giving 0.1 ft. to the minute, being subdivided clearly enough for a reading of 5″. This suggestion is good, and, under certain conditions, such a board could be used with profit, but there are practical reasons against it. One is that it is difficult to carry such a board, for a continued survey, especially if one is using pack mules, or even a wagon; the main objection, however, is that, when observing, the focus has to be changed from the mark to the star; and, when several observations are taken, in direct and reverse positions of the instrument, the change of focus causes delay and inaccuracy. The observation being taken during daylight, when Polaris or other circumpolar stars are invisible to the naked eye, its position has to be found by its hour angle, determined by the instrument with the aid of the vertical and horizontal circles. Furthermore, the proper sidereal focus has to be determined by an object as distant as possible. Otherwise, the star cannot be seen, even if the optical center of the telescope is coincident with its position in the heavens. Therefore, the focus plays as great a part in this observation as either of the co-ordinates defining its position. The hour angle method is an old one. The daylight observation, requiring no lights and ordinarily taken immediately after supper, comes at the most convenient period to give accurate results, and eliminates all danger of inaccuracy, being taken in daylight by the observer alone, without the aid of assistants.

In regard to the question of securing azimuth to a smaller reading than that of the least reading of the vernier of the engineer's field transit, which is 1 min., the writer is inclined to think that the least count of the vernier is as close as is necessary for these observations.

In regard to angular measurements for field procedure, practice has shown that the engineer's transit reading of minutes is sufficiently close. On account of terrestrial obstacles and weather conditions, there is a limit to the ability of man to project perfect lines on the ground. An angle, or azimuth, carefully calculated to seconds, or less, on paper, may develop an error of many minutes when applied to the earth's surface. This is the rule, not the exception. It would be impolite to mention specific instances. However, this does not imply that careless calculations should be made. A practical working limit should be used for field purposes, capable of rapid computation and application; and the tables and formulas in the paper have that end in view.

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In regard to the use of direct observations on the sun: Mr. Newbrough states, "by taking four can generally come within about a minute. To one who is accustomed to these, the calculation is very rapid." The example given in the paper, in regard to direct observations of the sun, is where a circumpolar star cannot be secured. Direct observations of the sun have been used for many years, but nearly every engineer has found that, on account of the sun's rapid motion, its intense brilliancy, and the difficulty in observing its limbs, it is hard to secure an accurate azimuth; and every field man has a natural fear that these observations are not as perfect as they should be. To obviate this, the example of equal altitudes of the sun is one which has a check, or proof, as to whether the results obtained are true. It will be noted that there are two series of observations, A. M. and P. M., which solve the celestial triangles and give independent azimuths. By bisecting the resultant angle to the south, which is obtained from the equal altitude observations coincident with the A. M. and P. M. triangles, another result is obtained. The sum of these three results, together with the difference of azimuth for the total hour angles from noon, should equal 360 degrees. Therefore, this furnishes a complete check on this method, and any error can be at once detected. This is the only solar observation that will check itself, and it is valuable on this account.

Mr. Newbrough observes:

"One of the most valuable features of this paper is the table for running east and west lines, which is a great improvement over the secant method."

Evidently, he has run innumerable miles on the western plains and in the mountains with a transit; and only one who has can appreciate the difficulty of running a rhumb line from a great circle line. In timber, when using the tangent method, the cutting is not along the rhumb line. (The rhumb line in this sense is either the parallel of latitude or a line of any constant bearing other than the meridian.) This cutting should mark the line on which the corners are laid, but in a few miles this tangent departs materially from the line sought, and confuses the land holder and those seeking to find their property locations. As previously stated, the secant is too complicated to give practical and accurate results, as too much dependence has to be put on assistants, who frequently are employed only for the time being and are necessarily new to the work. Too few engineers have realized that the rhumb line, and not the great circle or transit line, is the one on which the corners should be placed, and the line which the Courts hold, in cadastral surveys made by the Government, gives the boundary to deeded lands. Many textbooks on geodetic work give methods of finding the latitude and longitude of points on a line. They also give the back azimuth and forward bearing, but the writer

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can find no formula, which can be used in field practice, which will enable the engineer to locate this line by appropriate offsets from the great circle or transit line. The writer finds that many corners which should be on the rhumb line, as shown by the field notes, have been placed on the transit line. Table 1 will enable the locating engineer to place the corners where they belong; and the following formula will be sufficiently close, when frequent check observations are taken, and can be used where the table is not convenient:

$$D = 1.01 + \times \tan. \phi \times d^2 \times \sin. \alpha;$$

D = offset from tangent to rhumb line, in links;

ϕ = latitude of observer;

d = distance along tangent, in miles;

α = bearing of line.

Mr. LeBaron has evidently gone into this subject quite thoroughly. As suggested by him, most engineers take an incorrect view of the magnitude and difficulty of retracement-resurveys. At certain periods, they have handled other engineering works, and, naturally, assume the execution of cadastral surveys to be simpler and easier than the building of a railroad, a jetty, or other structural work. They generally view this subject from the monetary standpoint. When they design or construct a jetty which costs a few hundred thousand, or even a million, dollars, they are inclined to take a deprecating view of cadastral surveys. However, the writer is glad to know that such men as Messrs. LeBaron, A. M. Strong, James L. Davis, A. T. Parsons, H. V. Hinckley, and Leonard S. Smith, have given their attention to this important subject. This work is forcing itself on the attention, not only of engineers, but also of settlers and land owners in the West, and is assuming such proportions that the need of a definite method of prosecuting it has urged itself on the Engineering Profession. During the last five or six years, in Nebraska alone, the writer has had under resurvey about 125 townships, a conservative estimate of the value of which is about \$28 000 000; and, while this work has involved the homes and property rights of many thousands of people, he is glad to state that, by the use of the system described in the paper, no opposition has been raised by any of the parties affected thereby; and Nebraska, with its school lands, together with all the land holders interested, has cheerfully accepted the results determined by these methods.

Mr. LeBaron's discussion, in regard to the position taken by some lawyers concerning these questions, is exactly in line with the experience of nearly all engineers at one time or another. Very few lawyers wish to be involved in these land boundary cases. Many technical questions, other than those involved in law, are raised; and, as these have to be considered simultaneously, few who are well versed as lawyers have had the opportunity to consider all phases of the sub-

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ject. The old and well-seasoned lawyer, as a matter of fact, hesitates to take up such questions. It is mostly the young and inexperienced ones who "rush in where angels fear to tread;" and it is right here, probably, that Mr. LeBaron's experience with lawyers comes about. The average lawyer, as Mr. LeBaron states, when going into one of these cases, always brings out that threadbare question of the starting point, although the Courts, time and again, have declared that this is immaterial, so long as all original corners involved are properly used. There is one great truth running through all of this; namely, that the higher Courts have taken a most just view of the situation involving property rights and original surveys; and these decisions are the legal basis for our methods.

Mr. Strong's suggestion in regard to the failure of surveyors and engineers to use the proper length of chain is pertinent. The resultant hardships on property owners, which are seen in everyday practice in the West, are partly due to this lack of foresight; and the Desert survey of 1854, referred to by him, which has caused untold trouble and litigation, is the result of unthinking surveyors and engineers.

Mr. H. V. Hinckley suggests that the Dawes Commission, in allotting lands in the Indian Territory, established corners which the surveyors of the Interior Department did not use. The reason for this, evidently, is the United States statute which provides that all surveys and resurveys shall be made by the Commissioner of the General Land Office. Under the conditions stated, there were evidently two surveys, only one of which, of course, could be official; and that one would have to be designated by the statute.

Mr. James L. Davis considers the question of referring lines of land surveys to the true meridian. The writer certainly agrees with him on this point. Too many of the textbooks, especially those sold for the use of local surveyors whose training has not enabled them to use methods generally approved by engineers, have been inclined to misuse, or relegate to the rear, this important subject. The azimuth of a line is nearly as important as its measurement. The Courts, in nearly all decisions, hold measurement over alignment. There have been good reasons for this, however, the main one being that the early surveys were made with the 66-ft. Gunter's chain, which, if long or short, nearly always gave a constant error; and the alignment was secured mostly by the surveyor's open-sight compass, which gave the magnetic meridian by the aid of the needle. This resulted in an extremely rough and variable determination of azimuth, whether referring to true or magnetic meridians. It was also affected by local disturbances, such as local attraction, thunder-storms, the attraction of the sun, and various other factors. The Courts justly held that the measurements should hold over the courses, as secured by these methods, and on this was based the doctrine of proportional measurements. However, as

Mr. Sweitzer. the instruments and methods have improved wonderfully in the last few years, conditions at present have become reversed; and the writer thinks all engineers will agree that the azimuth or bearing of a line may be taken more closely in average rolling country than measurements can be made, in the same time, by the average chainman. However, the resurveys under discussion refer to the Court decisions in regard to proportional measurements, as executed by former surveys. The writer certainly concurs with Mr. Davis in his observations in regard to azimuth. His equation for equal altitudes of the sun appears to be the same as the writer's, but in a different form, checking the A. M. and P. M. triangles, in equal altitude observations. However, it seems to be somewhat simpler. The trouble with using the sun, as previously stated, is the apparent erratic motion, compared with the fixed stars, and the difficulty in securing the center. Therefore, in taking an observation on the sun, there should be a complete check, which cannot be secured by an independent observation; and the method suggested in the paper gives three complete checks, so that any error is immediately revealed. A sun observation should be taken only when a circumpolar star cannot be observed.

Mr. A. T. Parsons' observations in regard to the surveyor, with the best of intentions, surveying a plot adjacent to an established corner, without regard to other corners affecting this area, is an example of the cases which have been causing nearly all the disturbance throughout the West. This is partly due to the ignorance of the surveyor who does the work, and partly to the cupidity of the land holder, who does not wish to pay for executing the work properly. It is only a question of time when both suffer the consequences, and that is when the adjacent occupant wishes to secure his holdings under the original deed, acquired from the Government, giving him a portion of the legal subdivision of a section indicated by the original monuments, which, in the majority of cases, will conflict decidedly with such a survey.

Mr. Leonard S. Smith has added a most interesting discussion on the legal question involved in retracement-resurveys, has shown clearly and in detail the "significance of adverse possession," and has brought out clearly the different conditions involved in this complex question. He has also illustrated a case in which both azimuth and measurement are to be taken into consideration. In the last six or eight years, during which time retracement-resurveys have been under consideration in Nebraska, only once has this question been brought up, and that was in the relocation of the 5th Guide Meridian from the Kansas-Nebraska boundary, through Tp. 1, North. The question involved was that the original survey was complicated by a retracement made a year or so after, by a duly authorized deputy surveyor, whose plats had been legally accepted and by which title had passed; but they gave different angles than those recorded by the deputy who

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had run the original meridian. The method pursued in this retracement was the same as given by Mr. Smith, and the writer is very glad, indeed, that his survey has been sustained by the Wisconsin Court, as it can now be used as a precedent for the like method pursued in the Nebraska case. Mr. Smith's discussion, together with its comprehensive formulas and tables, is quite an addition to this subject.

In using latitudes and departures to secure the distances between nearest known original corners, it frequently happens, as in the case of a meridional line, that the line on which the corners are found departs considerably from the true meridian. In practice, when restoring the average Government survey, this does not so often happen, but, when it does, it is frequently considerable. In such cases errors have often been made by using the latitude for proportion, and not the resultant of the latitude and departure, being the direct distance between the corners. An illustration will probably make this clear: Suppose an attempt is made to restore 6 miles of interior corners, 1 mile east from the west range line of a township, namely, the corners to Sections 6, 7, 18, 19, 30, and 31. Assuming that all the corners on the west range line are in, or have been replaced, and that the northeast corner of Section 6 and the southeast corner of Section 31 are in the same condition; now, running west from the southeast corner of Section 31 to the range line, noting course and distance, thence along the range line to the northwest corner of the township, thence along the town line to the northeast corner of Section 6; tabulating this and securing the latitude and departure, gives the lost course and distance between the northeast corner of Section 6 and the southeast corner of Section 31, which is the resultant of the latitudes and departures, and not of the latitude, except when a line between the two corners in question does not depart from the true meridian. Now, this line can be proportioned from the original notes, north and south. However, the positions of the various indicated five interior section corners, dependent on the original survey, will be east or west of this line. Frequently, they will be considerably east or west, depending on the lengths given in the original plat or notes. Having secured their proper positions, a new chained length over all would have to be found; and this will necessitate a second proportion, as in most cases the east boundary of Sections 31, 30, 19, 18, and 7 are given as equal in the original field notes; and, therefore, they will have to be in proportion; and this naturally would have to be based on the final lengths secured. In practice, the case just given is exceptional, but it does happen; and it is well to remember this example and proportion the total length of a line and not simply the latitudes. Convergency, of course, should never be neglected, even in the case of a single section.

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Mr. Smith points out the fact that the positions of the original corners came remarkably close to where his calculations for azimuth and distances show they should have been. The writer has found this to be so in many instances. In the resurvey of central McPherson County, and other places in Nebraska, where exterior lines of townships were first resurveyed, using the existing original corners and proportioning in from the field notes the remaining corners, on these town and range lines, and using these exteriors as a basis for calculation to secure distances across the township to locate the interior section corners, proportioning in from the old notes, it was remarkable how close the original corners were to these proportional positions. In nearly every instance, the calculated positions were within a few links of where the original deputy had placed his corners. This would naturally be so, as the field notes are a detailed statement of the operations on the ground, reciting the course and distance; and only an accidental error would tend to disturb this harmony. These practical illustrations go to verify the correctness of the theory of proportional measurement, and justify the legal opinions handed down by the Courts in the method of restoring lost and obliterated corners where physical evidence of the original has disappeared, and where the testimony of "one who knows" the locus of the original corner cannot be secured.

Mr. Carpenter states that the writer "makes little mention of the use of proportional measurement in the relocation of a corner." The pamphlet on "Lost and Obliterated Corners" is valuable for locating one corner at a time, and, as most surveyors and engineers have been used to the methods as set forth in that pamphlet, it may be hard for them, on first reading the paper, fully to understand direct proportion by methods of closure based on the original notes. Lack of time has prevented the writer from entering into an academic discussion relative to the precedence of one line over another, and he has had to confine his remarks more to general theory aud legal points. Mr. Carpenter's discussion is to the point, but it is well to remember that, in the resurveys mentioned in the paper, the writer is relocating and re-establishing the original Government survey, and the Courts should be allowed to decide the question of the statute of limitations, whereby an individual may hold land other than that indicated by his Government patent, the restored original corners on the ground, or the proper relocation of corners according to the original plats and filed notes.

Summing up the discussion, it is apparent that all who have taken part are more or less of the same mind in regard to this subject. It is a peculiar thing that none of the textbooks on resurveys has considered it at any length. There is no doubt that, within the next few years, more weight will be given to it, and detailed methods will be formulated for considering these new questions.

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THE JUST VALUE OF MONOPOLIES, AND THE REGULATION OF THE PRICES OF THEIR PRODUCTS.

By Joseph Mayer, M. Am. Soc. C. E.

With Discussion by Messrs. P. L. Reed, Maurice G. Parsons, D. C. Serber, and Joseph Mayer.

Recent industrial development has produced a large number of enterprises, among them those supplying transportation, communication, light, heat, and power, which are to a considerable extent natural monopolies; that is, their products or services are sold at prices not governed by competition. Excessive prices have resulted therefrom. To secure fair prices to the public, various commissions have been established to control them, and many Court decisions have been rendered for the same purpose; but all these decisions have been insufficient to establish clearly the principles which should govern the determination of fair prices of monopolized products.

The Courts hold that a monopoly, if properly managed, is entitled to such prices for its products as will secure a fair profit on the just value of its property.

Thus far, no satisfactory method of ascertaining what is a fair profit and what is the just value of the property of a monopoly has been described by the Courts or commissions. The hopes and fears in regard to the probable future course of the Courts and commissions in their efforts to fix fair prices for the products and services of monopolies have large influence on the current market value of their securities, and are a factor seriously disturbing the even course of business. Therefore it is of importance to establish the principles from which the fair prices of monopolized products can be reliably deduced, and this is the purpose of the following investigation.

The difficulty in regard to what are fair prices arose with the disappearance of competition. The prices arising from reasonable competition are considered fair and just by all Courts and commissions. Just or fair prices of monopolized products, therefore, are such as would rule under fair competition, fair competition being such as gives the value of the product to the producer. The necessity for regulation arises from the prevalence of such prices as will give to monopolized industries unfair profits, or profits larger than prevail in competitive industries at the same time and place.

Just prices, therefore, are such as produce in monopolized and competitive industries the same rate of profit. To determine fair prices, therefore, we must ascertain the rate of profit in competitive and in the various monopolized industries. Profit is the compensation of the owner of an enterprise.

The prices of industrial products are equal to the material costs, or the expenses of production plus the profit of the owner. The expenses of production of the same product are different in every enterprise; the profit of the owner, therefore, is different to the same extent. The profits of a business depend partly on the capital invested, but still more on the intelligence and skill used in its original design and in the management thereafter. The vastly different rate of profit, or ratio of profit to capital invested, which prevails in industrial enterprises, enlarges the scope of activity of the most successful managers, and removes the incompetent ones; it is the only standard by which the ability of a manager can be accurately tested; it destroys altogether many ill-planned enterprises, and forces the remodeling of others; it is the source of progress in all industry, as it is the incentive enforcing the adoption of radical improvements in methods of production with all their expense, risk, and trouble; it is the reward for intelligent and the punishment for incompetent management. Even for the maintenance of the prevailing standard of economic efficiency, it is absolutely necessary that the scope of the enterprises of least cost of production and largest profits be constantly extended so that those of highest cost of production and smallest profit may be abandoned. The prevailing prices are equal to the cost of production plus the profit of the most inefficiently managed enterprises that succeed in existing, and the products of which are needed to make supply equal demand. Any regulation of prices which interferes seriously with this process of lowering the cost of production must lead, not only to stagnation, but to retrogression of economic efficiency. When higher than competitive prices are maintained in any industry, enterprises of exceptionally low standard of efficiency are thereby enabled to exist. The inefficient regulation of monopolies, permitting excessive prices of their products, therefore, often leads to a lowering of the standard of economic efficiency.

If the prices and profits of competitive enterprises are just, just prices for monopolized products must be such as will secure vastly different profits in different enterprises, according to the degree of intelligence used in planning and managing them. Any attempt to establish a uniform rate of profit on the physical valuation of the plant of monopolized enterprises, therefore, is radically wrong. Of two such plants of equal physical valuation, one may be that of a prosperous enterprise earning 20% dividends with just prices for its products, the other that of an enterprise on the eve of abandonment, because of ill-adjustment to the conditions of success. The value of one plant is not that of its parts, but can only be judged by its net earnings; that of the other is only a small fraction of its cost of reproduction less physical depreciation.

Since the profits of individual monopolized enterprises must remain vastly different to secure economy of production, the only profits which can and should be alike in competitive and monopolized industries are the average profits.

The prices of monopolized products or services, therefore, must be regulated so that the average profit of each monopolized industry is the same as that of competitive industries.

If it is practicable to ascertain the average profits of the various industries, this principle will enable us to judge the fairness, on the whole, of any definite proposed system of prices of monopolized products. It alone is quite inadequate to serve as a guide for creating such a system of prices. For this latter purpose it must be supplemented by a detailed study of the different industries in order to ascertain the just differences between the prices of the same product in different places, or when produced under such different material surrounding conditions as are independent of the skill of the management of each enterprise. The difficulties of ascertaining the average profits of the various monopolized and of competitive industries are apparently so great that this problem must first be solved in order to ascertain whether the proposed system of regulation is practicable. An exact definition of the term "average profit" must precede any further discussion.

In any large enterprise, carried on through a long period, money (or products and services of a value which can be expressed in money) is invested at various times, and interest and dividends, or other moneys, are paid out to the owners, at other times, as compensation for ownership. What is the profit or rate of profit in this enterprise? For the sake of brevity, the term "profit" will often be used in this paper for "rate of profit," which latter alone concerns us here. The rate of profit, when constant, is the ratio of annual compensation for ownership to the money invested. It has the same meaning as the rate of compound interest secured on the investment.

The term "average annual profit" of an enterprise from its beginning to the present may be defined as the present value of all the receipts of the investors, divided by the sum of the present values of the average capital of each year. For determining the present from the past values, the rate of interest taken must be the same as the rate of profit. The percentage of profit is the rate of profit multiplied by 100. If we call x the percentage of profit, and make $y = 1 + \frac{x}{100}$, then the above definition of the percentage of average annual profit leads to the following equation:

$$\frac{\Sigma\, a\, y^n}{\Sigma\, A\, (y^m + y^{n-1} + \ldots + y^{m-m_1+1})} = y - 1$$

where a represents a sum received by the investors; n the number of years which have elapsed since it was received; $a\, y^n$ is the present value of the sum, a; and the whole numerator, $\Sigma\, a\, y^n$, is the sum of the present values of all moneys received by the investors. A is the average capital of a year; m is the number of years which have elapsed since the middle of that year; m_1 is the number of years the capital remained in the enterprise; the whole denominator is the sum of the present values of the average capital of each year; and $y - 1$ is the rate of profit. The values of y and x can be found most readily by using compound interest tables; the calculation is simple, and is a purely algebraic problem which need not here be considered. The main difficulty lies in ascertaining, from the accounts of the corporation and other available sources, the correct amount of all the sums paid in, and received by the investors, with the dates of payment. If investors are executive officers, their salaries should be considered as expenses, not as compensation for ownership. Fees of directors are also expenses. Temporary loans made to the corporation are best not considered as part of the money invested in the corporation, and the interest on them is then part of the expenses. The rate of interest paid by a monopoly for temporary loans is not different from that paid by competitive corporations, and does not need regulation. Bondholders should not be considered as mere creditors; their compensation often largely depends on the success of the company. They must be regarded as investors; otherwise the amount of profit could be manipulated, and would largely depend on the amount of various kinds of securities issued. The interest going to bondholders must be considered as part of the profits. and the money paid in by them as part of the capital. The balanced average profit of several corporations is obtained by adding the numerators of the foregoing equation found for each of them and dividing this sum by the sum of the denominators for each of the corporations. With proper regulation of the corporations, which is required to secure the interests of the security holders, it will be possible in future to ascertain with reasonable accuracy all the facts needed to determine their average profit. The objection will be raised that the profit of a corporation is not what is paid out to bond and stockholders, but its net earnings. For the investor, the profits are what he receives divided by his investment, the investment being the amount paid to the company for his securities; and if the average profits as thus determined are—in the past, present, and future—the same in competitive and in monopolized industries, the investor is but justly treated, if he was in the company from its start. The public, it is true, pays for the net earnings, but if the investor cannot obtain more than a fair share of these net earnings, the public, with this system of regulation, will obtain in future what it misses at present. Furthermore, there is no reason to assume that monopolies will accumulate net earnings more rapidly than competitive enterprises. The annual net earnings of a corporation are the increase in value of its property during the year. The definition alone is sufficient to show the extreme difficulty and complication in determining the actual, not the nominal, net earnings. Any regulation based on net earnings, therefore, would have a very uncertain basis.

Practically the same objection may be raised in another form by stating that the profits of the investor consist, not only of the money received from the corporation, but of this money and the increase in market value of his securities. The market value of the securities is an estimate of the present value of the future net earnings.

The investor cannot collect now these future net earnings; he can only transfer his claims to others. If future profits are justly regulated, the public need not pay to the investor more than what is fair, and it pays then nothing for any increase in market values that may occur.

It will hardly be denied that justice to monopolized industries would have been secured if the principle of equal average profits on the real capital invested in competitive and in monopolized industries had been enforced from the start; and that with proper regulation of all corporations, such as is necessary for the protection of the interests of the stockholders, it would have been practicable to ascertain with reasonable accuracy all the facts necessary for such regulation. With old corporations, however, it is now impossible to obtain these facts for the remote past; and, if it were possible, the attempt to make the average total profits in the past and future of the different monopolies and of competitive industries the same, by reducing future profits below the average in industries with excessive past profits, and increasing future average profits where past profits were low, would work grave injustice among the investors of the present.

The closest practicable approach to justice will be secured by making the average future profits of present and future investors in monopolized and competitive industries the same. This will be most nearly attained when the present market value of the stocks and bonds outstanding is considered as the present capital of the corporations. The future capital can then be obtained by adding all the moneys received in exchange for stocks or bonds sold.

Short-time loans and other liabilities of corporations are best not considered as part of the capital, and the interest paid on them not as part of the profits. With this modification, the proposed system of regulation becomes practicable, and can be introduced without material disturbance of the relative value of the securities of monopolies and competitive industries, and, therefore, without injustice to present investors. To obtain a fair market value of securities, eliminating a large part of the accidental changes due to manipulation and other momentary causes of fluctuation, the average value during a few years should be taken. For many securities, especially of small corporations, there is no real market, because there are no sales; in these the value can be approximately estimated from the dividends and interest received, unless they are new enterprises which have not yet obtained sales commensurate with the capacity of their plants. It is not necessary to ascertain the capitalization of all the corporations in a monopolized industry. The average profit resulting from any proposed system of prices in any monopolized industry may be ascertained with reasonable accuracy when the capitalization and the sums received by the investors are known for the bulk of the corporations engaged in this industry. Corporations in which the value of the securities is not ascertainable, therefore, may be omitted from the calculation without serious error. After present capitalization is ascertained, it will be easier, with proper regulation of the accounts of corporations, to determine closely future capitalization. The amount of dividends and interest obtained by the security holders will also be easily ascertained.

As all additions to capital can be closely estimated, the proposed method of regulation will give just returns on all new investments of capital in monopolized industries. It may be claimed that this amounts to legalizing, for all future time, the present often unfair charges for monopolized products. This is not so. Where the charges of monopolies are known to be unfair, the investing public is aware of the fact and knows that they may be reduced by Courts or commissions. The market value of the securities of such a monopoly, and to some extent of all monopolies, is thereby reduced. This market value is the estimate of the present value of the probable future profits, making allowance for probable future regulation of prices. To allow the same average profits on the market value of the securities of monopolies and of competitive industries, therefore, is to allow to monopolies such profits as the public believes they will get, and means reducing their charges to the same extent as is, by the public, considered likely to happen. This is the closest practicable approach to justice. Absolute justice is impossible whenever injustice has been tolerated in the past and is then made illegal by a change of public opinion.

After having found a method of judging the justice on the whole of any proposed system of prices of the products of a monopolized industry, let us look at the method which must be followed in framing such a system. Let us take as illustration the gas industry. What we need is the just difference in the prices of gas of a given quality in different cities, and that in the prices of different qualities of gas. After we have these just differences we can frame a system of prices, starting with the existing prices in one city, which system is relatively just for different cities and different qualities of gas. If this system gives average profits in the gas industry, equal to the average profits in competitive industries at the same time and in the same place, it is not only relatively but absolutely just. If the profits are too high, all the prices must be lowered, without change in the differences; in the opposite case they must be increased. Just differences in price are those due to inequalities in the costs of production, caused by those differences in conditions which are independent of the degree of skill used in the design and management of the plants. In ascertaining the differences in cost of production, those methods of production must be assumed which are most generally used and therefore best known. Differences in cost of production due to either unusual skill or unusual incompetence of management must, in monopolized as in competitive industries, cause equal differences in profits and no difference in prices. Neglect of this principle and determining the just prices for individual enterprises by obtaining their costs of production and adding a standard profit would be disastrous to economy of production in all industries.

If, for a just regulation of prices, it were necessary to ascertain the cost of production of individual enterprises, and to determine the just profits of each enterprise from an estimate of the degree of skill shown in its design and management, then the just regulation of the prices of monopolized products would be utterly impracticable, because nobody is competent to perform the operation. The inevitable result would then be the public ownership of all monopolized industries.

To come back to our special instance: The just difference in the price of the same quality of gas in two different cities is the difference in cost of production plus profit, in the two cities, if the gas were made and distributed in both cities by the most widely used method of production. The rate of profit to be allowed is the average rate in competitive industries at the same time and place. The cost of gas consists of cost of manufacture and cost of distribution. An experienced engineer, familiar with the costs and methods of laying gas pipes, must study the conditions affecting costs in the two cities, and must find the difference in costs of distributing gas, per 1 000 cu. ft. sold.

The quantity sold per mile of pipe has evidently an important influence on this cost of distribution. Municipal ordinances, the nature of the pavements, rates of wages, and costs of pipe, all influence these costs. An engineer who makes many such estimates will be able to develop rules by which it will be a comparatively simple matter to judge the quantitative influence of the various causes affecting the costs. In a similar manner, the cost of manufacture and the average leakage must be investigated by engineers familiar with the subject. The fact that only one, and this the best known, method of manufacture need be investigated, greatly simplifies matters. Since only differences in costs between different cities are needed, the influence of all the factors which are the same in the cities compared can be neglected.

In a similar manner, all other monopolies must be investigated. Just prices for monopolized products can thereby be established without interfering with the economy of production.

The process of obtaining just prices of monopolized products is evidently expensive, and calls for commissions of experienced men of good judgment who have no other aim than justice.

Physical valuations of the property of monopolies, by determining the cost of reproduction less depreciation of their plants, are evidently of no use for this purpose. After just prices of monopolized products have been obtained, the just value of the securities of monopolies, or of the monopolies themselves, consists in the present value of their future dividends and interest payments. These can only be judged with reasonable accuracy after the just prices have been in force for some years.

Before this, market values, where such exist, are the only guide. Where these are absent, the value of monopolized property is unknown and practically unknowable, unless the principles by which the future commissions will be governed are accurately known. In this latter case their course of action can be approximately foreseen, and the future prices and the consequent profits estimated; from these the just value of the properties can be judged with nearly the same accuracy as that of competitive enterprises.

With the proposed regulation of prices of monopolized products, the monopolists are left free to find and apply the methods of production which will obtain the desired products with the least effort. Those enterprises which succeed in reducing costs of production by introducing new and more efficient methods obtain all the savings secured in the form of increased profits, until the new methods become widely known and increase the average profit in the industry, when the regulation steps in and reduces prices. If fair competition had existed, the increased profits would have caused increased supply and consequent reduction of prices, to the same extent.

With the proposed method of regulation, the rate of profit in a monopolized enterprise is a true measure of the efficiency of management, while, without regulation, it may be only a measure of the degree of injustice of the prices exacted. With this regulation, the owners of the enterprises have in the rate of profit a measure of the capacity of their managers, and know when it is time to advance or discharge them. Such regulation, therefore, will increase the economy of production by enlarging the scope of able and removing incapable managers; it will thereby be for the ultimate benefit of both the owners and the customers of monopolies. The injustice of excessive profits or inferior efficiency of monopolized enterprises is more and more felt to be intolerable, and will inevitably lead, either to just control of prices, or to public management of all monopolies.

Government Monopolies.

Judged by an economic standard, the various governments—National, State, and Municipal—are a network of huge monopolies of which we are all compulsory owners and customers. The total sum of the prices we all pay for their products is, as in private enterprises, equal to the total costs of production plus the total profits. The annual profits are equal to the increase in the value of the government property less the increase in the government debts. The net cost of the annual products is equal to the prices we pay less the annual profits. Since the profits, either positive or negative, belong to us, we are mainly interested in the cost of production, and not materially in the profits which come out of our pockets and afterward belong to us. In competitive industry and in private monopolies, justice to the consumer is obtained by either natural or artificial regulation of the prices of the products. In governmental activities, the net prices we pay necessarily equal the costs of production. Therefore, justice must be secured by regulating the cost of production. The only present attempt at such regulation is the periodical change of the managers. To secure real efficiency of production, we should have a reliable standard with which to measure the efficiency of our managers so that we may promote the capable and discharge the incapable ones. The best managers are those who secure for the desired quality of product the least cost of production. To obtain the best managers, and consequent economy of production, of all the services and products furnished us by governments, it is necessary, therefore, to ascertain, in every department of government action, the exact cost of production of all the products or services supplied. Only after, in every department of a large number of cities or States, the costs of production of every service or product have been ascertained and scientifically compared, by allowing for the quantitative effect of all those causes of difference of costs, with the most usual methods of production, which are independent of the skill of management, is it possible to judge reliably, and with the needed accuracy, the degree of economy of management of any department, or city, or State.

Thus far, the accounts of governments have not been kept in such manner as to enable one to ascertain correctly the cost of production per unit of any of the services rendered. The interest charges on all capital used, whether represented by debt or not, must, for rational comparisons, be included as part of the costs of production. Where comparison is desired with the prices of services rendered in other places by private monopolies, the taxes which a department would have to pay, if privately owned, must be included as part of the cost of production. All the costs of production, per unit, for every government service or product, should be determined and all the quantities measured. The resultant total costs plus the profits should agree with the total payments by the consumers. The citizens, or owners, should be informed of the unit costs of all the products and services supplied.

As we need commissions to regulate justly the prices of the products of private monopolies, so do we need commissions to ascertain the just prices, or the costs with average efficiency of production, of the products and services of government monopolies. The just prices for any service or product of a government monopoly may be defined as those which, on an average, cover the cost of production, and which vary in time and place by amounts equal to the differences in cost of production resulting from causes independent of the skill of management.

No private business of large size with many departments could be carried on successfully without accurate cost-keeping in every department, and without changing the management where the resulting profits are unsatisfactory.

The scientific comparisons of the costs of production of the various government services and products cannot be made by individuals. A well-organized body of capable, experienced men is necessary, especially for ascertaining the quantitative effect of all those causes of differences of cost of production by the most common methods, which are independent of the efficiency of management and determine the just difference in costs or prices of different cities or States.

After this work has been accomplished, the reputation of public administrators will for the first time agree with the facts, and it will then be possible to treat justly our most important public servants. At present the relative merits of men, institutions and economic experiments can at most be judged qualitatively; a quantitative estimate and a quantitative social science will only become possible after the just and the actual prices of all the products and services furnished by governments are known. It must be conceded, however, that the difficulties of such comparisons are in some departments very great. Those departments where the quality of the services or products supplied can be measured most accurately, or where the products are uniform in different places, will, other things being equal, most naturally be taken up first, and after ample experience in these, others may be attacked.

The establishment and maintenance by commissions, of such prices, for the products of all private and government monopolies, as would prevail under free and fair competition, and the maintenance of such compensation, where it is practicable and economical, is a necessary part of the administration of justice, and is, therefore, pre-eminently a government function.

The regulation here advocated is just, is compatible with the highest efficiency of production, and greatly facilitates progress. With such regulation, Municipal, State, or National franchises need only prescribe the quality of service desired, stating that such prices will be allowed as will secure the same profits as would prevail under free and fair competition. The savings due to invention and progress of every kind with this regulation, after a short time, as in competitive industries, accrue to the public; and, without injustice, the public can at any time change the quality of service demanded or the rate of taxation of the monopoly, because such changes will not affect its profits.

The objection will be raised that competition inevitably results in inadequate wages and lack of employment of the inefficient; that, therefore, to extend, in effect, the range of free competition to all the activities of man will lead to the permanent misery of a large class of the population; and that the proposed system of regulation is thereby condemned as unjust.

Justice consists in creating and maintaining such relations among men as will produce universal welfare. The existence of a large class of miserable people, therefore, is a sure indication of injustice. The regulation proposed would indeed be condemned if it implied the permanent existence of such a class. This class can only be helped by the will and the power of the more successful. The proposed regulation increases their wealth and the consequent power to assist those who cannot now be made efficient, and to provide the means for training the rest. The inefficiency is due partly to lack of proper training, partly to unsuitable ancestors. The only radical remedy, therefore, is an improvement in the methods of selecting the ancestors for the future generations and the training of their children. To achieve these results, increased wealth is most essential, and the best chance, even for the inefficient, lies in maintaining the most efficient system of production of wealth, which calls for industrial rewards in proportion to efficiency, and in directing by public opinion and legislation the humane impulses of the wealthy toward the use of their power to the creation of universal industrial efficiency.

The proposed regulation of the prices of monopolized products and services requires the creation of State commissions in all the States, with power to enforce fair competition where practicable and useful, and to fix and maintain such prices for all monopolized products as will make the average profits of every monopolized industry, embracing many independent enterprises, the same as those of the competitive industries at the same time and place.

These, or separate commissions, should also ascertain the just prices for all products and services furnished by municipalities. These commissions must have power to prescribe and enforce for all municipalities such a system of cost-keeping as will determine correctly the unit cost of every product or service supplied.

National commissions with similar powers must be established to fix the prices for the monopolized industries engaged in interstate commerce and to ascertain the just prices for the services and products supplied by the States.

To ascertain the efficiency of the National governments, international commissions will be needed to find the just, and the actual, prices of all services and products furnished by them. Only in this way can the efficiency of all the departments of such governments be measured. The best managed departments and governments will thereby be pointed out; they can then be studied, and their best features adopted.

The difficulty of the work of these commissions increases with the size of the field of their activity and with large variations in the qualities of the services rendered, especially where quality is of supreme importance and where its value cannot be measured accurately. The regulation by State commissions of the prices of the products and services furnished by private monopolies and the fixing of the just prices for similar services and products furnished by municipalities is most easily accomplished. The fixing of the just prices for such immaterial services as public education is most difficult, but not impossible. It cannot be well denied that a well-argued valuation of the public education of all the States by the foremost educators of the country would give important clues leading to the general improvement of public education. Such a valuation, however, would evidently be the most difficult work that could be given to a National commission.

It would not be hard to determine the degree of difficulty of the various kinds of work which must ultimately be done by such commissions. Other things being equal, the easier work should be undertaken first; but, as other things are not equal, the great importance of some of the proper work of such commissions requires it to be undertaken before much easier and simpler work is commenced. Where the absence of regulation does most harm, it must be undertaken, even where the difficulties are so great that complete success is impossible, since approximate justice is better than gross injustice.

The amount of human effort that is beyond the range of competition is constantly increasing the direct establishment and maintenance of just prices for all the products of private monopolies, and the indirect enforcing of such by ascertaining and publishing the actual and the just prices of all the services and products supplied by governments, is, therefore, an increasingly important part of the administration of justice.

The choice of the managers, of every grade, in all organized human effort, is the most influential factor in determining success or failure, increase or deterioration of efficiency, progress or retrogression.

For the best choice, a reliable standard of past, present, and future ability is a necessary condition. With the regulation proposed, private and public monopolies can, for the first time, at all times, measure the real ability of their managers by the profits in the first, and by the costs of production as compared with the just prices in the latter. The best managed enterprises are thereby reliably pointed out, and their methods will spread rapidly. The ablest managers will rapidly obtain larger scope for their ability, and the incapable ones will be made harmless by discharge, in the same manner as in competitive enterprises. The just reproach of lack of economy and progress in monopolized enterprises will then to a large extent disappear.

The vast advantages secured by just control of the prices of monopolized products and services, therefore, fully justify the large amount of difficult work required from men of highest ability and the inevitably large expenses incurred to provide it.

Charging What the Traffic Will Bear.

There are still occasional advocates of the justice of the principle of charging what the traffic will bear; it is of importance, therefore, to ascertain the difference between the prices resulting from this principle and from the regulation here proposed. Assuming monopolists intelligently pursuing maximum profits and not fearing the loss of their monopolies, the resulting prices of their products may be calculated.

Let us assume that with p, the present price per unit of a monopolized product, the demand for it is q, the cost per unit, c; then the total profit is $(p - c)\, q$.

If the competitive or just price of this product is p, and if a reduction, dp, in the unit price increases the demand by dq, then it is profitable to make this reduction when the consequent increase of profit is larger than the competitive profit on the additional quantity produced.

The new profit is $(q + dq)\,(p - dp - c)$. The old profit was $q\,(p - c)$. The increase of profit, therefore, is

$$(q + dq)\,(p - dp - c) - q\,(p - c) = (p - c)\,dq - qdp.$$

The competitive profit on the additional quantity is $(p_1 - c)\,dq$. The price reduction, therefore, is profitable when $(p - c)\,dq - qdp > (p_1 - c)\,dq$, or when $(p - p_1)\,dq > qdp$, or when $p - p_1 > q\,\frac{dp}{dq}$.

$p - p_1$ is the difference between monopoly price and just or competitive price.

q is the quantity of demand, and $\frac{dp}{dq}$ is the ratio of reduction in unit price, to change in quantity of demand resulting therefrom. As long as this inequality exists, it is profitable to reduce the price. The price, therefore, will be reduced until $p - p_1 = q\,\frac{dp}{dq}$, or $p = p_1 + q\,\frac{dp}{dq}$.

The most profitable monopoly price, p, therefore, is larger than the competitive or just price by $q\,\frac{dp}{dq}$. This difference between monopoly and competitive price is largest when a given change in price causes but small change in amount of demand, which is the case with very useful or necessary products.

It is smallest when a given change in price causes a large change in demand, which is the case with all luxuries which become accessible to larger numbers of consumers by a reduction in price.

The difference between monopoly and competitive price is also proportional to the demand; it is largest for articles of general consumption and small for luxuries.

It is evident, therefore, that uncontrolled monopolies are especially severe on the poor; and that monopoly prices are always higher than competitive prices.

When the monopolist fears the loss of his monopoly, he may reduce the price below the most profitable figure.

DISCUSSION

Mr. Reed.

P. L. Reed, M. Am. Soc. C. E. (by letter).—This paper discusses a subject which everybody who follows political and social tendencies at all must realize to be among the most important and vital, and concerning which there has recently been, and will continue to be, much consideration and discussion. It is a comparatively new form of governmental activity, entered into with general reluctance and only because its difficulties seemed less than those which had grown up naturally around unrestricted industrial development.

In the author's analysis and recommendations for the control of monopolies, he retains those incentives which make for individual efficiency and economy, for which free competition is usually given credit, while making fully available the collective economies which are credited to combinations and consolidations. At the same time the rights of the consumer are fully maintained. This is an ideal which is rarely attempted, and is undoubtedly attractive. It is believed that the general principles will be found sound—sounder than those commonly advanced, which imply either prices fixed by free or even compulsory competition, or, on the other hand, by the limiting or fixing of profits at an arbitrary percentage of the capital invested.

The writer, however, considers that to attempt to reduce these principles to algebraic equations inevitably brings in too many complications to lead to entirely successful results, and that, in addition to this difficulty, there are what seem to be defects in the author's mathematical treatment to which attention should be invited in the discussion, in order that his main arguments may not be weakened.

In expressing algebraically the percentage of profit, the factors, y^n, in the numerator, and ($y^m + y^{m-1} +$, etc.) in the denominator, are introduced to make allowance, in determining the annual average rate of profit, for delays or postponements in the receipt or declaration of these profits or dividends, and they serve no other purpose. In this case, therefore, y is a function of the rate of interest which these profits might be expected to have earned for the investor had they been distributed earlier. This does not necessarily agree with the average annual rate of profit of the enterprise, and would only do so by chance, unless the profits were immediately reinvested in the same property. On the left side of the equation, therefore, y does not represent the same quantity as y on the right side, and it would seem that the expression would be much simplified and improved, if, for the unknown y on the left side, were substituted the same function of a fixed rate of interest such as the investor may be assumed to receive on his liquid funds, or the legal rate of interest.

As previously suggested, there are practical complications which would interfere so seriously with the use of such a formula as to make it of doubtful practical application, such as "going value," "depreciation," "obsolescence," and various suspense accounts. These factors would prevent to such an extent the exact determinations of the values, a and A, as to make the consideration of interest on delayed profits an unreasonable refinement. Mr. Reed.

Again, under the heading, "Charging What the Traffic Will Bear," an equation is given (in the fourth paragraph) for the increase in profit due to a decrease in price with corresponding increase in quantity, assuming the cost per unit to remain constant. This equation is wholly independent of the "competitive" or "just" price, and is complete as it stands. The reason for bringing the latter in is not understood, since the most profitable monopoly price bears no relation to it. The most profitable monopoly price may evidently be more than, equal to, or less than, the competitive price, depending entirely on what effect a given change in price bears to the corresponding change in quantity. Still assuming the total cost per unit to be constant, the relation shown by the equation just referred to may be expressed in this way (each change in price to be relatively small): It is profitable to make a reduction in price when the ratio of a proposed change in price to the present profit per unit is less than the ratio of the resulting increase in quantity sold to the present quantity sold.

It is again suggested that such mathematical conclusions are of doubtful practical application. In this case the cost per unit rarely remains constant, usually lowering as the quantity or output increases. Furthermore, it is exceedingly difficult to predict, even approximately, what change in quantity or sales will result from a proposed change in price.

Not only does it appear that the desired conclusions cannot be obtained by mathematical formulas, but neither can they be obtained by the application of even less fixed and definite rules. In attempting to determine such rules, the author is led into what seems to be an inconsistency, or circle of reasoning. Thus, the allowable average rate of profit of a monopoly should be the same as the average rate of profit in a similar competitive industry. The value of an existing monopoly shall be taken as the market value of its securities; but the average profit of such a monopoly, based on the market value of its securities, is already the same as the average profit in similar competitive industries. There is nothing to be done. We end where we started.

The author recognizes this objection, in the following:

"It may be claimed that this amounts to legalizing, for all future time, the present often unfair charges for monopolized products. This is not so. Where the charges of monopolies are known to be unfair, the investing public is aware of the fact and knows that they may be reduced by Courts or commissions. The market value of the securi-

Mr. Reed. ties of such a monopoly, and to some extent of all monopolies, is thereby reduced."

If the proposed rule for fixing the prices of a monopolized product is based on assumed previous knowledge of investors, Courts, or commissions of what fair prices should be, may not one ask by what rule the investors, etc., are to obtain their knowledge; and, if they already have a good rule, of what use is another which can only tell them what they already know?

It is believed, therefore, that the author's attempt to avoid physical valuations will not be generally considered successful, and that the attention which has been paid to such valuations to assist in the determination of fair prices is not without good reason.

Probably little more can be stated as a specific rule than in the words of the author:

"To fix and maintain such prices for all monopolized products as will make the average profits of every monopolized industry, * * * the same as those of the competitive industries at the same time and place."

The practical difficulties are plain enough, and perfect justice is unattainable, but it requires no special gift of foresight to realize that these difficulties must be met, that decisions must be reached, and that this will be working along the line of least resistance from the present stage of our industrial development.

Mr. Parsons. MAURICE G. PARSONS, JUN. AM. SOC. C. E. (by letter).—This paper is timely, although it follows closely the papers entitled "The Going Value of Water-Works"* and "The Valuation of Public Service Corporation Property."† The questions discussed are of present political and business importance, but the principles used in handling them are neither entirely established nor very widely spread. Academic economists throw but little light on their practical solution, leaving the engineer largely to his own resources, therefore a wide discussion of monopolies and prices, by engineers, and from many viewpoints, is highly desirable.

There is to-day "a sound of a going in the tops of the mulberry trees," new to this generation. We are emerging from the factory system and entering the trust age—the age of more complete division of labor, of combination, of artificial and natural monopoly. Competition in certain branches of business seems to be dead, to have yielded to natural monopoly. Without going into the question of whether we will soon have monopoly in all activities, it may be stated with confidence that natural monopoly is here to stay for some time. Competition, with its wasteful duplication of plant and management, has been found uselessly expensive. Competition, with its combinations—con-

* *Transactions*, Am. Soc. C. E., Vol LXXIII, p. 326.

† *Transactions*, Am. Soc. C. E., Vol. LXXII, p. 1.

spiracies against the public—has been found more apparent than real. The natural monopoly is the inevitable and welcome method of supplying many present-day needs. Mr. Parsons.

When the people realized that monopolies were not only being benefited by the advantages obtained by the elimination of competition, but were also artificially raising prices, various remedies for this unjustness began to be suggested. Taxation of net income, taxation of gross income, rate regulation, and Government ownership are a few of the many schemes having for their object a fair adjustment between cost of production, prices, and income. All aim—some directly, some indirectly—at rates.

The author has discussed the most fundamental part of the problem, which is also the most difficult phase to solve, namely, the determination of a fair rate of profit. Much thought has been expended, but, in the writer's opinion, without reaching a satisfactory solution.

First, it would seem that Mr. Mayer has failed to grasp the fundamental difference between competitive concerns and natural monopolies. Generally, more risk is involved in competition, due to ability or trickery on the part of other concerns, than in natural monopoly. A monopoly risks only the welfare and patronage of the people; a competitive concern risks also the dangers of competition. Competitive industries must have duplicate plant and management, thereby entailing, in general, expenses greater than those of a monopoly doing the same amount of business. A monopoly is often the creature of the people, originating in an exclusive franchise. For those reasons may not one ask whether a profit or a price fair under competition is necessarily fair under monopoly?

The second trouble with the author's solution may be defined as one composed of a mixture of doubtful reasoning and difficulty of application. Of what avail will it be in any individual case to know average prices and profits if "the profits of individual monopolized enterprises must remain vastly different to secure economy of production"? Will it not be as difficult "to ascertain the just differences between the prices of the same product in different places" as it will be, without a knowledge of average prices, simply to determine just prices? A question might be raised as to whether it is right to assume that competitive profits are fair rather than exorbitant or low, since many competitive concerns have grown unduly wealthy and others have failed. It might be fully as hard to decide whether competitive profits were fair as it would be to determine, in any given case, a fair monopoly profit. Certainly competition—when, indeed, there is real competition—is influenced by monopoly in fixing prices. Again, what would one do, under this method, were he asked to fix a monopoly price for an article not produced by competitive establishments? The Interstate Commerce Commission has had difficulties in its endeavor to

Mr. Parsons. enforce fair competition, but the labors of this body would be small compared with those of commissions detailed to determine average profits for all kinds of local, State, and National organizations. Even after this average profit has been determined, the engineer must deviate therefrom in any given case.

A third question arises after a study of this paper, namely, the author's method of evaluations. If a monopoly is valued by its stock, one must, to be just, squeeze out the water. To waive the privilege of a physical valuation, as affecting prices, but to establish the amount of fair securities, after knowing what a fair price will bring in, seems rather like a merry-go-round to those who hold that, in a measure, fair prices are determined by some kind of evaluation. Of course, we no longer hold to the labor theory of value, but we can admit the plant as a factor in value: To pay a man for time given, whether to a necessary or a useless object, is one thing; to hold that a natural monopoly, supplying a public necessity necessarily only with the aid of a large investment, is entitled to rates influenced in some degree by the relative amount of capital tied up, is another. Evaluating a business by capitalizing its profits is looking at things from the investor's point of view rather than as one who seeks to establish a fair rate.

To the writer, a rational method of dealing with the entire problem is that outlined below. This has fair rates, and its object is the gradual lowering of these rates, to be obtained indirectly rather than directly. We cannot arbitrarily say that 50-cent gas is just—perhaps it is and perhaps it is not. The subject must be approached by a roundabout path.

The first step is the determination of a fair percentage of profit on the investment. Money sunk must be protected. Money sunk in a risky venture is entitled to more reward than that invested in a sure proposition. Money put in necessary enlargements does not deserve as high a return as the original capital used in developing the business. Pioneer monopolies, treading unknown ground, should be more highly rewarded than those following a beaten path. The just percentage of net income—the fair profit—may differ with each case, but can it not be as well established arbitrarily as by any other method? This is a hard question to settle, the one of fair profit, but it is fundamental. Why not weigh the risk and say that 5, 10, or 20% is fair to a natural monopoly, rather than become involved in average competitive profits?

The consumer is directly concerned with two things: Prices, which are frequently an object of criticism; and adequacy of service, which generally escapes notice unless it deteriorates. Certain it is that natural monopolies should be regulated so as to secure for the public an adequate and economical service. The word adequate is meant to be used in a literal sense, thereby excluding luxury of service. A simple illustration will suffice: Water companies should furnish ample water without uselessly expensive, uneconomical works. At the same time, it is wrong to saddle a community with rates to pay for a plant of ten times the required capacity. We must have just enough and just the right kind of service, economically furnished. Mr. Parsons.

The third factor in fixing a just rate, the actual cost of production, is one involving an array of engineering problems. None of these can be solved, except by rough guess, unless thorough and accurate accounting and physical record systems are kept. The development and physical value costs are two of these problems. Depreciation and sinking fund expenses can be but approximate. "Who can tell what a day will bring forth?" Still there are cases when these factors have been supposedly worked out to a cent. Operating expenses are more easily ascertainable, even though the economic expenditures in this direction are extremely complex in some cases. The cost of production, as determined by all its factors, should be a just one. It is not right to recoup too rapidly for development outlays, nor should the next generation pay the depreciation charges of to-day. In a growing community the natural monopoly should be regulated so as to secure proper extensions. These should come when they are needed and should be economically secured.

Guided by these factors—fair profit, adequate service, cost of production, extensions—the fixing of rates becomes a simple matter. We approach the question of rates indirectly. We should enforce administration policies to secure, not the lowest possible rate of the moment, but the long-time, economical, low rate.

Where will all this regulation end? Perhaps, finally, it will go no further than regulation. Perhaps we shall have Government ownership of all natural monopolies. Perhaps we shall have Government ownership of all business. Business and government are, in the last analysis, on the same foundation; both exist for, and by virtue of, the people. If the Government does take over all business, we have to go still deeper and see who runs the Government—whether it be the people or the money—but with this the Engineer is not professionally concerned. He has, however, a duty as a citizen at the polls. If the people want anything long enough and sincerely enough, they can get it. The Engineer's duty at the polls, however, is a minor one; his big duty of citizenship is at the tiller of business. What is needed is not men with their ears to the ground, nor men who endeavor to stave off this or that public or private opinion. Absolute justice is, at best, difficult to obtain. We, therefore, need engineers who will play fair; who will keep faith with both sides; who will steer a straight course; who will exercise good business sense; and who will solve our problems.

D. C. Serber, M. Am. Soc. C. E. (by letter).—The regulation of prices of products of monopolies was recently advocated publicly before an official committee in the United States Senate by two well- Mr. Serber.

Mr. Serber. known representatives of the trusts, Judge Gary and Andrew Carnegie. It has aroused great interest, not only on account of the prominence of its two advocates, but mainly because it touched on a subject of unusually great public interest, namely, the social and economic unrest now being manifested all over the world.

We are entering on a new phase in the evolution of our economic life. Free and unlimited competition seems to have completed its work and is about to give way to concentration of effort in production. While competition regulates market prices according to the law of supply and demand, concentration of effort makes it necessary to find some other basis for fixing prices, because the supply is regulated and limited.

Mr. Mayer has recognized this necessity, and is of the opinion that the Government should regulate prices. This he proposes should be done according to a formula which he has derived, and by which he believes "just" prices could be fixed.

He states that the market value of securities of a monopoly is the estimate of the present value of the future profits. To allow, therefore, profit on the market value of securities of monopolies is to allow to monopolies such profits as the public believes they will get. This is the closest possible approach to justice. Absolute justice is impossible wherever injustice has been tolerated in the past and is then made illegal by a change of public opinion.

Would it be right to accept the "present value of market securities" as an "estimate of the probable future profits"? What is the present "value" of securities? Is it their market price? Nothing under the sun is as changeable as that. Prices of products of monopolies fixed on this basis in August, 1907, for instance, would have been from three to five times more than the prices of those products fixed by the prices of securities in August, 1909. In fact, the market prices of stocks change every day; they depend on the relative strength of the "bears" and the "bulls," but are by no means a measure of the worth of a monopoly or its products.

Mr. Mayer evidently has used the term "value" instead of "price." Political economy gives the definitions of these terms. An attempt will be made in this discussion to give their ratio.

Things can be valuable to us on account of some inherent or latent desirable feature, by means of which we are capable of satisfying one or more of our wants, as, for instance, food, clothes, air, etc. This is the intrinsic value or use value of a thing. On the other hand, things may have a value to us, even if we do not need them for our own use, when they can be exchanged for objects which we need. This is the exchange value of a thing. The use value depends entirely on our need, the degree of our culture, our taste, habits, etc. A steel bar has more value than a gold one for reinforced concrete.

The definition of exchange value is not as obvious. It does not depend on our wants or habits. Thus, in the preceding example, the gold bar can be exchanged for a far greater number of useful things than the iron bar, although the latter is much more valuable for concrete. Free air, on the other hand, cannot be exchanged for anything at all, while compressed air can. The exchange value of a thing, therefore, is independent of the individuality of its possessor. Compressed air has an exchange value because some useful work has been spent on the free air in order to convert it into compressed air. The more useful work spent, the higher will be the pressure of the air and the greater its exchange value. It is the same as the principle of conservation of energy. If equal amounts of work are spent on the production of two different commodities their exchange values are equal. Political economy defines exchange value as the amount of useful effort necessary for the changing of raw material into a finished product. Mr. Serber.

Hence, values are created by natural growth. They are naturally fixed by the effort spent on their production, and they are permanent and stable until human genius succeeds in reducing the amount of effort needed for producing them. Prices, on the contrary, are arbitrary expressions of these values, and are naturally extremely unstable, as the arbitrary element in them continually changes. The actual exchange values of things are entirely obscured by their market prices. So many foreign considerations are at work in transforming the real expression of man's work that it cannot be recognized. Interest, profit, rental, expense of hunting for markets (the real element of competition), cheating, greed, speculation, politics, are some of the forces that fix market prices. Of all things, stocks are endowed with the most changeable prices; their values are completely hidden. Sometimes securities with good exchange value are reduced in price, while others with absolutely no value have a high price, and these prices change with lightning rapidity. How, then, would Mr. Mayer take the market prices of "securities of monopolies" as a "just" criterion for the real worth of those monopolies? With these prices of "securities" as a measure, prices of commodities produced by the monopolies would have to be changed hourly, or at the will of the "bears" and the "bulls," who, instead of the Government, would, in fact, "regulate" the prices.

We are awakening to the fact that something is wrong in our economic relations, and that something must soon be done to remedy it. Regulation of prices by the Government is recommended as a sure cure. As this "regulation" is urged in the name of "justice," how should prices be determined in order that they may be "just"? Whether the Government could regulate them, so as to satisfy the demands of justice, will follow from that as a corollary.

Mr. Serber. The writer will attempt to investigate analytically the average way in which prices of products are made, and perhaps that will indicate what elements in the make-up of prices should be tampered with, in order to make them really "just," and how far this regulation ought to be carried to be of benefit to anybody but to the monopolies themselves.

It will be necessary to make a number of assumptions as the analysis progresses, and arbitrary quantities will also have to be used. These can be verified or changed by anybody who does not agree to them, and a different result will be obtained. What is proposed is a method of analysis in which the arbitrary quantities are used only for the purpose of illustration, though the writer will endeavor to assume those quantities as close to actual conditions in life as possible. The determination of their true numerical value would require a close study of a great many industries.

Man is so constituted by Nature that, in order to exist, he must consume different things, just like any other animal; but, unlike most other animals, he must produce the things he consumes. The process of production consists in taking raw material, as it is found in Nature, and adapting it to his various needs. Clay, for instance, is used in producing pottery, for building a puddle wall, for manufacturing cement, or for making brick. In other words, raw material, through the exertion of a certain amount of effort, is converted into something which has not only a use value, but also a definite exchange value. If the producer does not need it for his own use, he can always find some other fellow creature who could make use of it, and would be expected to give in exchange for it other values, the production of which required an amount of effort exactly equal to that consumed in its production. When this is done, and only then, "justice" will be attained. A "just" price would be one which expresses in dollars and cents the exact exchange value of any useful object produced by human effort. If the producer does not get in return the full equivalent of the value which he offers for exchange in the market, somebody else must have retained the missing portion on its way to him during the process of exchange, that is, during its passage from the producer to the consumer. This is not an assumption, but a self-evident truth.

Let M = The cost of the raw material as it exists in Nature, before any human effort has been made to procure it or conserve it. This would be, for instance, the cost of coal and oil fields, primitive forests, mines, quarries, land, etc.

E = Total expense incurred in transforming the raw material into something useful.

lE = Item of labor in E.

bE = Cost of maintenance and deterioration of buildings, including rental. Mr. Serber.

mE = Deterioration of machinery.

aE = Cost of finding new markets, such as expenses of commercial travelers, salesmen, agencies, advertising, etc.

tE = Taxes.

TE = Cost of transportation.

l, b, m, a, t, and T are fractions, or percentages, of E, that is, ratios of the parts of E to E.

S = Interest on the investment.

r = Rate of interest.

p = Profit.

R = Rate of profit.

C = Total capital invested, or cost of product to the producer.

P = Price of product, or its cost to the consumer.

V = Actual exchange value of the product.

N = Number of independent stages through which the product passes before it reaches the consumer.

Our economic relations are such that no product passes from the producer directly to the consumer without intermediate steps. As a rule, we buy our necessaries in retail stores, which, in turn, get their stock from wholesale houses, and these depend for their supply on manufacturers. The latter may use as raw material something which in itself is a finished product and which has had to go through several stages of production before it reached them in the necessary state. Take as an example the clothes we wear:

(1) We buy them in a store.

(2) The storekeeper gets them from a wholesale tailoring establishment.

(3) The wholesaler gets the work done by individual tailors at their homes.

(4) The cloth, which is itself a finished product, is made in mills.

(5) The wool for the cloth is supplied by a trading company.

(6) This material is supplied to the traders by sheep raisers.

(7) To raise the sheep properly, irrigated and fertilized pastures are needed, which are purchased by the sheep raisers from some former landowner.

Thus far, seven stages have been followed. One or several stages may have been omitted, such as commission merchants or wholesale dealers, or by combining the stages of the manufacture of the cloth itself into only one stage; also, the land might have changed owner-

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It is impossible to imagine any product, however simple, which has not passed through several consecutive and independent stages of production during its process of evolution.

From the adopted notation:

$$C = M + E \quad \dots\dots (1)$$
$$E = (l + t + b + m + T + a)\,E \quad \dots\dots (2)$$
$$s = r\,C\,;\; p = R\,(C + r\,C\,) = R\,C\,(1 + r) \quad \dots\dots (3)$$
$$P = C + s + p = C + r\,C + R\,C\,(1 + r) = C\,(1 + r)\,(1 + R) \quad \dots\dots (4)$$

Let the rates of interest and profit be assumed as constant; also let the items belonging to different stages be denoted by corresponding subscripts, P_1, P_2,..P_n, C_1, C_2,..C_n, E_1, E_2,..E_n, etc. The cost of the material, M, may vary with the number of stages, as in every stage some new material may be added. However, as the first stage usually starts out with most of the material, it will be sufficiently correct to assume that M is constant and equal to the sum of the costs of all the material which might have been used during all the stages.

The producer in the first stage starts with the outlay, M, he spends E_1, his total investment is C_1, and he sells the product for P_1. Hence,

$$C_1 = M + E_1\,;\; P_1 = C_1\,(1 + r)\,(1 + R) = (M + E_1)\,(1 + r)\,(1 + R.)$$

The producer in the second stage starts with an outlay of P_1, and, therefore,

$$C_2 = P_1 + E_2 = C_1\,(1 + r)\,(1 + R) + E_2\,;\; P_2 = C_2\,(1 + r)\,(1 + R)$$
$$= C_1\,(1 + r)^2\,(1 + R)^2 + E_2\,(1 + r)\,(1 + R).$$

Similarly,

$$C_3 = P_2 + E_3 = C_1\,(1 + r)^2\,(1 + R)^2 + E_2\,(1 + r)\,(1 + R) + E_3.$$

. . . .

$$C_n = P_{n-1} + E_n = C_1\,(1 + r)^{n-1}\,(1 + R)^{n-1} + E_2\,(1 + r)^{n-2}\,(1 + R)^{n-2} + \;.\;.\;. + E_{n-1}\,(1 + r)\,(1 + R) + E_n \quad \dots\dots (4\text{-A})$$

$$P_n = C_n\,(1 + r)\,(1 + R) = C_1\,(1 + r)^n\,(1 + R)^n + E_2\,(1 + r)^{n-1}\,(1 + R)^{n-1} + \;.\;.\;. + E_{n-1}\,(1 + r)^2\,(1 + R)^2 + E_n\,(1 + r)\,(1 + R) \quad \dots\dots (5)$$

In Equation 5, P_n is the price paid by the consumer for the finished product. In order to get a concrete idea of the variation of

P_n, numerical values will be substituted for n, r, and R, in the general equation. As the usual rate of interest varies from 5 to 7%, r will be assumed at 6 per cent. The rate of profit, naturally, has a greater range of variation, and depends on more conditions than does interest. It may run from 10 to 100%, and even more. The minimum will be assumed, and, therefore, $R = 10$ per cent. It has been shown that the number of independent stages of production, n, is variable. An example was cited of seven or more stages. Some products may pass through ten or twelve stages, while others may have only four. A conservative assumption will be $n = 6$. Mr. Serber.

Substituting these numerical values of n, r, and R in Equation 5, and writing for C_1 its equivalent, $M + E_1$, we get:

$$P_6 = 2.51\ M + 2.51\ E_1 + 2.16\ E_2 + 1.85\ E_3 + 1.58\ E_4 + 1.36\ E_5 + 1.17\ E_6 \quad \ldots\ldots (6)$$

The quantity, E, is variable in the same industry, and is irregular at that. In some industries it is greater in the early stages, in others it may be smaller. Let E_p be such an average, that, when substituted in Equation 6, it would give the same value of P_6, as would be obtained by assigning E, E_2, E_3, E_4, E_5, and E_6, their proper values. Then:

$$P_6 = 2.51\ M + 10.63\ E_p \quad \ldots\ldots (7)$$

In this equation M is the part of the price caused by charges for raw material which man's hand had never touched. Any effort made, or expenses incurred on it, before the seemingly real process of production has begun—as taxes, interest, and profit due to ownership of property, or to change of ownership, etc.—would enter into Equation 5 as one or more stages, E, depending on how many profit-bearing transactions have been made with it. M is a charge for material as it exists in the bowels of the earth, or in the depths of the waters, or in the thick of the jungle, but which is untouched, primitive, without man having ever raised a finger to exploit it. E_p in Equation 7 is the expense incurred by the producer during only one stage, exclusive of his outlay for material. Were the price of a product merely the cost of production and of the material, the expression for P_6, assuming $n = 6$, and $E_p' =$ the average for E, would be

$$P'_n = M + n\ E'_p;\ \text{and}\ P'_6 = M + 6\ E'_p \quad \ldots\ldots (8)$$

If, however, the popular conception of justice would allow the producer interest and profit only on his addition, E, to the value of the product, then:

$$P''_n = M + n\ E'_p\ (1 + r + R);\ \text{and}\ P''_6 = M + 6.96\ E'_p \quad \ldots (9)$$

Reference is here made to popular opinion, because Mr. Mayer bases his theory on "public beliefs" and "public opinion."

A comparison of Equations 7 and 9 shows that, apparently, the present price, P_6, paid by the consumer, is nearly twice the price,

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P_6'', which popular opinion would consider nearer to justice. The reason for this lies in the fact, that, at present, the producer in each stage charges interest and profit, not only on the effort E_p' which he himself exerts, but also on the efforts which all his predecessors have made during the preceding stages of production. More than that, he charges profit on his predecessor's profit, as well as on the profit which his predecessor's predecessor made on his own predecessor's profit; in other words, the profit is compounded. He also charges a profit on all the taxes which had been paid prior to his stage of production, and this is also compounded. He charges a profit on the cost of the raw material, which is also compounded. What has been said about profit is also true of interest. In fact, every item of cost of production grows with the number of stages at the rate of compound interest. The greater n, the number of stages, is, the greater are the total profit and interest charges levied on the actual value of the finished product. Therefore, when

$$\left.\begin{array}{l} n = 7,\ P_7 = 2.93\ M + 13.56\ E_p, \\ n = 8,\ P_8 = 3.42\ M + 16.98\ E_p, \text{ or } P_8 = 3\ P_8'' \text{ (nearly)} \\ n = 9,\ P_9 = 4.19\ M + 21.17\ E_p, \\ n = 10,\ P_{10} = 4.64\ M + 25.81\ E_p, \text{or } P_{10} = 4\ P_{10}'' \text{ (nearly)} \end{array}\right\} \ldots (10)$$

The comparison of P and P'' in Equation 10 is based on "public belief"; but "beliefs" in general are very poor arguments in analysis, no less poor than, for instance, if the fact of the rotation of the earth were established by a public vote. Also, the "public" until lately "believed" partly in free trade, partly in protection, as a panacea for all its economic ills. The quantity, E_p, will be examined more closely. From Equation 2 we have

$$E_p = (l_p + t_p + b_p + m_p + T_p + a_p)\ E_p \ldots\ldots\ldots\ldots (11)$$

In this expression the labor cost, l_p, and the taxes, t_p, are the only quantities which are not wrapped in a heavy mantle of compound profit obtained as an heirloom from the preceding stages of production. Labor alone (brain and muscle) offers its services to production during any stage, without demanding any profit or interest on itself, while taxes on property naturally cannot contain any element of profit or interest. Not so with the remaining elements of E_p. For instance, m_p is the cost of the deterioration of machinery. It is supposed to be a small fraction of the total cost of the machine added to the expense of production, so that, after a certain lapse of time, equal to the length of the period of usefulness of the machine, the sum total of these small additions would be equal to the original cost of the machine; but the machine purchased by the producer is in itself a finished product, the price of which should be obtained in precisely the same way as P_n was derived in Equation 5. The same is true about b_p, the maintenance of buildings, and about T_p, the cost of transportation. The construction of new buildings, or parts of buildings, or alterations or repairs, no matter how small, require material, labor, tools and machinery, transportation, interest on invested capital, profit, etc. Also building and operating railroads or steamships, or even trucks, require the same elements. They are finished and complete products in themselves. Therefore the parts, b_p, d_p, T_p, and a_p, of E_p, should be derived in precisely the same way as P_n was obtained. Hence the expression of P_n in Equation 5 can be modified still further. Mr. Serber.

Imagine an economic state of affairs such that the product goes directly from the producer to the consumer. This would not mean that there is only one stage of production, but that all the stages have been combined under one ownership. Then the total effort exerted on the material would be $6E_p'$, and the cost of the product to the producer would be $C_n''' = M + nE_p'$, or $C_6''' = M + 6E_p'$. The term E_p' will be different from E_p of Equation 5, because its elements, b_p', m_p', T_p', and a_p', will differ from b_p, m_p, T_p, and a_p. If interest and profit were now charged by the producer, the price would be

$$P_n''' = (M + n\,E_p')\,(1 + r)\,(1 + R);\text{ and } P_6''' = 1.17\,M + 7\,E_p' \ldots (12)$$

If this is compared with Equation 9, it would appear that these two expressions for price are very nearly identical; but, as a matter of fact, they are different, for the following reason: the quantity, a, which is the expense of finding markets, would entirely disappear from E_p' in Equation 12, because now the producer would not have to hunt for the consumer; the latter would rather have to look for the former, because there would be only one producer. The elimination of a is of great importance.

The relative value of the constituent parts of E_p may be taken as follows:

$$\begin{aligned}
l_p &= 20\% \text{ of } E_p,\\
b_p &= 2.5\% \text{ of } E_p,\\
t_p &= 20\% \text{ of } m - 0.5\% \text{ of } E_p,\\
m_p &= 2\% \text{ of } E_p,\\
T_p &= 50\% \text{ of } E_p,\\
a_p &= 25\% \text{ of } E_p.
\end{aligned}$$

These numerical values are mere assumptions, but express the average conditions, as nearly as possible. That $T_p = 50\%$ may seem excessive, but it should be remembered that the average prices of the entire national production are being considered. If P_6, for instance, is the price per ton of steel structures, it is intended to mean the average price of all steel structures built in the United States during a certain period of time. Some material has to be transported across the continent, and in that case, very often, the cost is fully 100%, if not more, not only of E, but of the cost of the entire finished

Mr. Serber. product. Nor would 25%, as the numerical value of a, be out of the ordinary. If the products are not disposed of in sufficient quantity, production will cease, and with it the profit to the producer. Hence consumers must be found by all "means," especially if the cost of those "means" is to be ultimately charged to the consumers, and with interest at that. When it is remembered that this item includes the expenses of advertising, commercial travelers, agents, commission houses, brokers, all sorts of "go-betweens," in short, a whole army of busy men, who produce nothing real; men who add nothing to either the use value, or exchange value of things; men whose efforts could be easily spared without in the least affecting the true value of the output of our national production; when it is also remembered that these "go-betweens" are mostly enjoying excellent incomes, which surpass many times the returns on the "profitless" efforts of the actual wealth producers, of the contributors of muscle and brain—when all this is borne in mind, it will not seem at all out of proportion to make $a = 25\%$ and $l = 20$ per cent. Some traveling men make as much as $25 000 yearly. Commission merchants, insurance and real estate brokers, Wall Street operators, contractors, etc., with their long retinue of legal talent—these alone enjoy yearly incomes which perhaps equal, or even surpass, the entire wage of the wealth producers.

Therefore, the assumed percentages will be considered as very nearly correct;

If a_p is eliminated from Equation 12, we get:

$$P_n''' = (M + n) \times 0.75\, E_p''' \,(1 + r)\,(1 + R)\,;$$

$$\text{and } P_6''' = 1.17\, M + 5.25\, E_p''' \ldots\ldots\ldots\ldots \quad (13)$$

where $E_p''' = (l_p''' + t_p''' + b_p''' + m_p''' + T_p''')$, and E_p''' is much smaller than either E_p or E_p', because b_p''', m_p''', and T_p''' are also purged of their element, a. A comparison of the prices, P_6 and P_6''', derived from Equations 7 and 13, would show that P_6 is more than twice P_6'''. In other words, the concentration of production alone and the elimination of competition should be expected to reduce prices to less than one-half of their present magnitude.

Now, a further step will be taken, and it will be assumed that, not only has the concentration of ownership penetrated into every branch of industrial life, but that all branches of production are merged in one great industrial enterprise under national control. In other words, it will be supposed that all the natural resources of the country have reverted back to the nation, and that the nation itself is the sole exploiter of these resources, the sole producer of all our wealth, of all our use values. In other words, let it be imagined, that by some approved method, the nation can produce and adequately distribute among its constituents all that is now being produced and distributed

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by either competing or combined, but independent, profit compounding, industrial units. Everything, in such a case, is owned and done by the nation, and for itself. The producer is the consumer, and the product no longer goes from the former to the latter, but from the former to himself. The nation owns all the farm lands, pastures, mines, quarries, forests, lakes; it produces all necessaries for life, as well as all machinery and implements required in the production of those necessaries; it owns the railroads, steamships, in fact, all means of transportation; and keeps retail stores, where the finished product is ready to be delivered to the consumer. Under such conditions, what would become of P and its equivalent? The incentive for production at present is gain. As this stimulus would cease to exist, profit and interest would be eliminated entirely, while no charges would be made for the raw material. The quantity, E, would still retain its partials, l, m, d, t, and T (the item, a, having been eliminated by the concentration of production, as was seen before), but, of these, only the labor cost and the taxes would be unaffected, while the remaining part would be greatly reduced, because these would now also be purged of profit, interest, cost of raw material, and expense of hunting for new markets; in other words, b, m, and T would now express exclusively the amounts of effort exerted on their production. Therefore, E would now also represent the magnitude of the true effort spent during one stage of production. This will be called E_v, and the subscript, v, will be used for all its items as well as for P, then:

$$E_v = (l_v + t_v + b_v + m_v + T_v)\, E_v \ldots\ldots\ldots\ldots \quad (14)$$

Also calling P_v the price at the end of the nth stage, that is, the cost to the consumer, we get:

$$P_v = n\, E_v = n\, (l_v + t_v + b_v + m_v + T_v)\, E_v \ldots\ldots\ldots \quad (15)$$

As nE_v represents only human effort absolutely needed in the production, it follows that P_v is also the equivalent of creative human effort, and, therefore, the price to the consumer would be equivalent to the exchange value of the product, or $P_v = V$.

Now compare Equations 5 and 15, that is, the present price of products with their exchange value. In order to do this it is necessary to find the ratios between each item of E_p and the corresponding item of E_v, as well as between E_p and E_v. From Equations 2 and 14 we get:

$$E_p = (l_p + t_p)\, E_p + (b_p + m_p + T_p)\, E_p + a_p\, E_p \ldots\ldots \quad (16)$$

and

$$E_v = (l_v + t_v)\, E_v + (b_v + m_v + T_v)\, E_v \ldots\ldots\ldots\ldots\ldots \quad (17)$$

As the labor cost and taxes remain the same, it follows, that

$$(l_p + t_p)\, E_p = (l_v + t_v)\, E_v \ldots\ldots\ldots\ldots\ldots\ldots \quad (18)$$

Also, since $T_p = 50\,\%\, E_p$, and $b_p = 2.5\,\%\, E_p$, and $m_p = 2\,\%\, E_p$, we can write:

$$T_p : b_p : m_p = 20 : 1 : 0.8, \text{ or } T_p + b_p + m_p = 21.8\, b_p \ldots . \quad (19)$$

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We have seen that the cost items of buildings, machinery, and transportation are at present obtained in the same way as P_n in Equation 5; therefore:

$$\frac{T_p\,E_p}{T_v\,E_v}=\frac{b_p\,E_p}{b_v\,E_v}=\frac{m_p\,E_p}{m_v\,E_v}=\frac{P_n}{P_v}\quad\ldots\ldots\ldots\ldots\ldots\ (20)$$

Hence, also: $T_v : b_v : m_v = 20 : 1 : 0.8$, and $T_v + b_v + m_v = 21.8\ b_v$. (21)

Substituting Equations 21 and 18 in Equation 17 we get:

$$E_v = (l_p + t_p)\ E_p + 21.8\ b_v\ E_v,$$

$$\text{or}\qquad 1 - 21.8\ b_v = (l_p + t_p)\,\frac{E_p}{E_v}\quad\ldots\ldots\ldots\ldots\ldots\ (22)$$

From Equation 20, making $\dfrac{E_p}{E_v} = K$

$$\frac{b_p}{b_v}\,\frac{E_p}{E_v}=\frac{b_p}{b_v}\ K=\frac{P_n}{P_v}\quad\ldots\ldots\ldots\ldots\ldots\ (23)$$

As wholesale houses, trading companies, mercantile establishments, and in general, business transactions, are eliminated in obtaining P_v, the number of stages of production, n, will be greatly reduced. However, it will be conservatively assumed that on the average there will now be five stages instead of six, as assumed in the case of P_n. Hence,

$$P_v = 5\ E_v\quad\ldots\ldots\ldots\ldots\ldots\ (24)$$

Hence, substituting from Equations 7 and 24 in Equation 23,

$$\frac{P_6}{P_v}=\frac{b_p}{b_v}\ K=\frac{2.51\ M + 10.63\ E_p}{5\ E_v}\quad\ldots\ldots\ldots\ (25)$$

$$\left.\begin{array}{l}
\text{When } M = 0,\ \dfrac{P_6}{P_v} = 2.1\ K\text{; hence } \dfrac{b_p}{b_v} = 2.1\\[2ex]
\text{When } M = E_p,\ \dfrac{P_6}{P_v} = 2.6\ K\text{; hence } \dfrac{b_p}{b_v} = 2.6\\[2ex]
\text{When } M = 2\,E_p,\ \dfrac{P_6}{P_v} = 3.1\ K\text{; hence } \dfrac{b_p}{b_v} = 3.1
\end{array}\right\}\ldots\ldots\ (26)$$

Substituting in Equation 22 the numerical values of l_p and t_p,

$$1 - 21.8\ b_v = 0.205\ K\quad\ldots\ldots\ldots\ldots\ldots\ (27)$$

Solving Equations 26 and 27 for K, and remembering that the numerical value of $b_p = 0.025$, we get,

$$\left.\begin{array}{l}
\text{When } M = 0,\quad K = 3.66\text{; hence } P_6 = 39\ E_v\text{, and } \dfrac{P_6}{P_v} = 8\\[2ex]
\text{When } M = E_p.\ \ K = 3.86\text{; hence } P_6 = 51\ E_v\text{, and } \dfrac{P_6}{P_v} = 10\\[2ex]
\text{When } M = 2\,E_p,\ K = 4.02\text{; hence } P_6 = 63\ E_v\text{, and } \dfrac{P_6}{P_v} = 13
\end{array}\right\}\ (28)$$

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In a similar way, Equation 28 could be continued to include the ratio of P_6 to P_v, corresponding to any ratio of M to E_p, or, for any cost of raw material. As there are no products in which no charge whatever is made for raw material, M can never be made equal to 0; on the other hand, quite often, the raw material is very expensive in comparison with E_p. Therefore, it will be extremely conservative to assume that, ordinarily, $M = E_p$ and hence, usually,

$$P_6 = 10\ P_v = 10\ V \ldots\ldots\ldots\ldots\ldots\ldots (29)$$

This equation shows that present prices of products are at least ten times their exchange values, or, in other words, only one-tenth of what we pay for things nowadays represents actual human effort, while nine-tenths of our expenses, or nine times our effort in creating wealth, represents contributions which we are forced to pay partly as profit and interest and partly to all sorts of "go-betweens."

Of these nine-tenths of our created wealth we are deprived legitimately; but, who can tell how much of the remaining one-tenth we are compelled to lose through illegitimate methods of "business"? Do we get full measure in our retail or wholesale establishments? Or do we get the use value of the quality for which we pay? "Business" levies another considerable share of wealth by underweighing, undermeasuring, adulterating the quality of the products and similar means. At first glance, it might seem incredible that "monopolies" (which are worth hundreds of millions, do business worth billions of dollars, are at liberty to fix high prices, and get enormous profits anyway) would stoop to such methods; it would seem that competition was the real cause for such low acts, because individual competitors were compelled to resort to such means, in order to be able to exist in the face of great competition which demanded low profits or even none at all. As a matter of fact, however, "monopolies" beat small competitors in dishonesty, because they practically have the field to themselves. It is gain, and the facility to obtain it through organization, that causes it.

The people are wondering why prices keep on rising. It is extremely puzzling that while technical improvements, discoveries, inventions, unprecedented progress in science, etc., ought to cheapen production to an almost incredible extent, not only do we not pay less for its products, but, on the contrary, they are more expensive; and it is bound to be so. In the first place, in many cases, the cost of raw material increases because it changes ownership very often. This means a greater numerical value for n in Equation 7 and a larger P_n, because M and n are larger.

Turning again to Equation 5, it is seen that probably both M and n are constantly increasing with the number of business transactions. E_p should become smaller with the improvement in production. Its elemental items are probably affected as follows: l, though it is dimin-

Mr. Serber. ished quantitatively on account of continuous introduction of machinery, increases qualitatively, as wages are continually on the increase, though to a smaller degree than the former, and hence, on the whole, l decreases; t is too small, and its variation is negligible; a is on the increase, especially during the period of transition from the competitive system of production to that of concentration; the monopolies must "hustle" to swallow their small adversaries; b, m, and T ought to be considerably smaller on account of the greatly increased output which they deliver. As the only increasing element of E is a, which is only about 25% of E, while all the other elements diminish, it is only natural to assume that E diminishes as methods of production are improved. M and n, on the other hand, though increasing in their numerical value, cannot be the only cause of the increase in P_n, because their variation is limited. Hence the real reason that prices are getting higher instead of lower must be the increase of the only remaining factor, $(1 + r)$ $(1 + R)$, and as r is practically constant, most of the change in P_n is due to the rate of profit, R, especially when it is noted that a comparatively small increase in R changes P_n considerably.

Though the cost of production has been greatly reduced in all industries, the consumer does not share in this reduction.

Three equations have been derived for the determination of price:

1st.—Equation 5 shows the derivation of price as it is formed at present. The numerical expression of this is given in Equation 7.

2d.—Equation 13 gives the price as it should be fixed by monopolies, when all stages of production of any one finished product are concentrated under one ownership, and when the claims arising through that ownership are satisfied through a charge of interest and profit.

3d.—Equation 15 is the general formula for price as it would be under national ownership of all natural resources, as well as of all known means of production and transportation. The numerical value of this is given in Equation 24, and a comparison between this and the equation for prices as fixed at present is given in Equation 29.

Which of these three methods would Mr. Mayer recommend to the Government as the one which is consistent with his idea of "justice" and with his idea of "just prices"?

Equation 5 is evidently not to his liking, as it expresses the very conditions which have called forth his recommendation for reform. The realization of Equation 13 is impossible, as we know that there is nothing in this world strong enough to keep within bounds of honesty, decency, and propriety the greed of monopolies backed by unlimited resources. Attempts have been made by the Government to differentiate between bad trusts and good ones; but, can there be any difference between them when the essence of the very existence of any trust is greed and gain? None of the existing trusts has at any time voluntarily reduced prices, though their gains are enormous, while to compel them to do so seems to be impossible; and yet, combination of production and concentration of capital are absolutely inevitable. Competition cannot and need not be revived. The trend of evolution points to expansion of concentration to include all branches of industry. Economical production of wealth demands it. It is in the interest of ourselves as the supposed owners of the great "per capita" wealth, about which we read with pride in the reports of the Department of Commerce and Labor. Mr. Serber.

Evolution in production demands monopoly and combination, as it demanded competition one hundred years ago, and trusts, therefore, are here to stay. To try to eliminate them would be both unpractical and futile, as no sooner would a monopoly be dissolved, than it would "reorganize"—unless our liberties were curtailed too much.

Industrial life has two great branches, production and distribution. The former creates wealth, while the latter disburses it, so that each individual atom of our social unit gets a certain (or rather uncertain, at present) share of that wealth, if any at all. Distribution, by the way, need not be mistaken for transportation, which is an item of production. It has been seen that production is passing through different periods of evolution and that competition gives way to concentration. Production creates exchange values, and concentration helps to reduce considerably the effort needed for creating them. Distribution, on the other hand, creates prices, by which the producer adds an increment to the produced value in the shape of profit. The ratio between this increment and the real value determines what part of the value should go to the producer. When competition was in vogue, production (as well as distribution) was controlled, to a certain extent, by the law of supply and demand. The larger the number of competitors in a certain line of production, the cheaper were the prices, and therefore the smaller the producer's profit.

Concentration, however, places both branches of industry in the hands of the monopolies. They can produce as they want and when they want; and can stop producing when they see fit to do so. They can withdraw the money from the "market," and create panics which bring misery and desolation to thousands. They can say to the man with the muscle or to the man with the brains, "Thou shalt not produce," and their order is implicitly carried out, though the man with the muscle and the man with the brain can produce, wants to produce, and must produce, in order to live. In short, they have unlimited sway over production in all its phases. The same is true with reference

Mr. Serber. to distribution. Equation 29 shows that, at present, prices are at least ten times the real value of the product. It is also seen that the factors which cause this are the profit, p, interest, s, and the share of the "go-betweens," aE_p, in each stage. The ratio of the latter to E_p is 25%; hence, in six stages, this share will amount to 1.5 E_p. Substituting from Equation 28, $E_p = 3.86\ E_v$, we get the "go-between's" share, nearly 6 E_v, or 1.2 P_v.

This means that the final price paid by the consumer to the producer is divided, or distributed in the following way: one part goes to compensate the value of the product, P_v, 1.2 parts are allotted to the "go-betweens," and the insignificant balance of 7.8 parts go to the owner of the implements, but not as compensation for the deterioration of these implements, for this element has been included in the items, b and m, of P_v. This lion's share is the accrued compound interest and profit which the producer sets aside for himself. It must be noted here, that the word "producer" is used, not to mean the person who exerts the effort needed in creating values, not the man with the muscle or brain—the latter had his recognition in the formulas through the modest item, l. The word "producer" here means the owner of the implements, the manufacturer, the monopoly, or the trust.

This producer has sole jurisdiction over the distribution of the finished product. He has full and unlimited control over it and over the very life of the man with the muscle and the man with the brain, by fixing prices at his pleasure. The writer does not refer here to the employer's right to fix the wages of his employees. The argument refers to something broader than that. It shows the unlimited control of the producer over the distribution of the entire increment of the national wealth, and it also shows that he distributes it in such a way as not to ignore himself altogether.

Justice cannot be obtained, prices cannot be just, unless they are fixed according to Equation 15. On the other hand, this will never take place, unless control over both production and distribution has been transferred from the present producers to the nation itself.

Mr. Mayer. JOSEPH MAYER, M. AM. SOC. C. E. (by letter).—The criticisms submitted make it plain that some of the writer's ideas have not been presented with sufficient clearness to convince the reader of their truth. The writer's fault lies partly in inadequate fullness of definitions, and this has caused a consequent confusion of the critics.

The fundamental ideas discussed in the paper are value, justice, monopoly, and regulation of prices. To avoid confusion, it is necessary to use each of these terms with a definite meaning.

Value.—Man wills to live. Things which further life are useful. Useful things which are difficult to obtain are valuable. Usefulness and difficulty to obtain are the two necessary elements of value, but, as both are essential, neither can be used as a measure of value. The value of a thing is measured by the quantity of other valuable things which can be obtained for it in free exchange. Values are extremely variable until there is a market where many purchasers and sellers meet. Things are exchanged for money; the amount of money a thing sells for is its price. When all prices at a given time and place are known, all the values are known. As long as the value of money remains the same, the prices are correct measures of values. Things have three prices, the bid, the asked, and the price of actual transactions. The last lies somewhere between the former two. The man who is obliged to sell will accept the bid price, and the man who is obliged to buy will pay the asked price. These prices differ widely where there are but few sellers and purchasers of a commodity. Speculators, who watch the markets, buy from urgent sellers and sell to urgent buyers, reduce these differences, and are thereby useful. The usefulness of a thing gives the upper limit of the price bid. For things produced in large quantities, by many independent producers, cost of production by common methods plus ordinary profits gives the average price asked. The usefulness of a means of production consists in furnishing a net income to the owner in the present and during a limited future. The price paid for a means of production approximates the estimated present value of the net income it will give to the owner. Mr. Mayer.

The net earnings of regulated monopolies depend on the prices permitted for their products and their cost of production. The values of these monopolies depend on these net earnings and the rate of interest, which latter must be known in order to determine the present value of future net earnings. The cost of production of the plant of a monopoly gives no indication of its value unless it influences either the present or the future regulation of the prices of its products. The value of a plant for making competitive products depends also on the present value of its future net earnings. This is largely influenced by the cost of production, with ordinary ability, not of a similar plant, but of one making similar products and securing net earnings of equal present value. The value of such a plant is governed mainly by its capacity to secure a low cost of production for valuable products. It depends at least as much on its location and the arrangement of its parts as on their cost, as much on the design as on the quantity and quality of its material parts. The value of the whole, therefore, is something entirely different from that of the parts or their cost of production. A complicated and old plant may be misplaced for present conditions, and will often contain many features ill adapted to the present most economic methods of production, and other features, the result of eliminations of unsuitable uneconomic arrangements, which may make it, if it has had able managers in the past, a much more economic plant than a new

Mr. Mayer. one of equal cost designed by a manager of but ordinary ability. A plant is largely a mental product, and its value depends on the quality of the designers' minds. This quality cannot be measured by a physical valuation based on cost of production or reproduction; it is only measurable by the net earnings secured. This is felt by all who attempt to ascertain the value of an industrial enterprise by a physical valuation. The absurd results obtained by such a valuation make it necessary to correct them by introducing other elements, such as depreciation, going value, intangible value, franchise value, etc. All these can only be estimated by comparing the present value of all its estimated future net earnings with that found by a mere physical valuation. The difference is the supplementary value of the many names, which, added to the physical value, gives again the real value used in finding the supplementary value. The physical valuation, therefore, is entirely superfluous.

Depreciation is sometimes defined independently as a physical quality, without considering the net earnings secured by the plant or by that part of it. The average useful life of a machine or building is estimated, and the present value is ascertained by subtracting from the value when new a rate of annual loss which will reduce it to nothing when the machine is worn out. A machine usually becomes worthless, not by what happens in it, but by what happens outside of it; it becomes antiquated because of new inventions and methods of production at different rates; and it is often evident that a machine or building has become antiquated and has lost value much more rapidly or slowly than previously estimated. It would be misleading to stick to a valuation based on uniform rules after new facts which prove them incorrect are known.

The importance of the new facts can only be estimated by the change of net earnings which they cause. The rule given for determining depreciation, therefore, is either untrue or dependent on future net earnings. The statement that the value of an industrial plant or a business is equal to its cost of production less depreciation, or to the cost of reproduction of a new plant of the same kind less the physical depreciation of the old one, therefore, is untrue unless additional elements of an immaterial nature are introduced, which practically change it to the rule that its value is equal to the present value of its future net earnings. For enterprises, the securities of which are largely dealt in on the exchanges, the quotations give the best available estimate of this value. They are the value, because they give the price for which known fractions of the total value of the business are sold.

To change by regulation the future prices of the products of an industrial enterprise, and, consequently, its net earnings, is to change its value, and an attempt to justify such regulation by trying to determine the value of industrial enterprises independently of their future net earnings is entirely futile. The fact that regulation changes and determines the value of industrial enterprises must be acknowledged; and the necessity and justice of the regulation of the prices of monopolized products must be established without denying patent facts. Mr. Mayer.

The method of valuation proposed by the writer is substantially that which is generally followed in valuing real estate; here, however, it is more difficult to ascertain reliably the terms of the sales taking place, since they are not published like the transactions on the stock exchanges. The valuer of real estate takes the known sales, compares the present and the estimated future net earnings of the properties sold, and determines that rate of interest which makes the value of the net earnings equal to the price obtained. He then ascertains the present and the estimated future net earnings of the property, the value of which is sought, and calculates, with the same rate of interest, the present value of all its earnings; this is the value of the property. The cost of production cannot be used where old properties are concerned, and especially where the property considered is of an uncommon kind, as practically all complicated industrial plants are.

The proposed method of regulation of prices of monopolized products uses only the sum of the values of a large number of plants manufacturing one product and the sum of their dividend and interest payments to the security holders, and then determines the price of this product; so that the average profit in this industry as thus determined is the same as that in competitive industries at the same time and in the same places. If the same method is used in valuing competitive and monopolistic enterprises, the errors in both, if the principle of the method is correct, will generally be of the same kind, and individual errors in both directions will largely balance, so that no injustice results.

When, as prescribed by some State laws, the net earnings allowed to individual enterprises are measured by their estimated value, then it is of the utmost importance that every single estimate is correct, and, in this case, every enterprise has a strong incentive to have its plant valued highly. In new States, where one has to deal only with new enterprises, a just valuation can be obtained by taking as the value of every business the amount of money paid in; and a just regulation can be based on values thus obtained. For old States the paper proposes one valuation based on average market values for the last few years. All future values would then be obtained from this first and only valuation plus the amounts of capital paid in thereafter. The value of all future enterprises, for the purpose of rate-making, would be determined by the capital paid in. The fluctuations

Mr. Mayer. of the stock market would be ignored and would have no influence on the prices of monopolized products.

Justice.—Hazy and fluctuating ideas of justice are the main causes of confusion in arguments on economic questions.

Justice consists in the establishment and maintenance of such relations among men as will secure the welfare of man. Man wills to live; that which furthers life is useful. Compensation in proportion to usefulness will secure the largest amount of usefulness; it will, therefore, most further life, and thereby will secure the welfare of man. By compensation in proportion to usefulness will the most useful be given the means of multiplying their kind and the useless prevented from so doing. Only thus can an efficient and vigorous society be secured and maintained.

The use of wealth as a tool for the production of wealth greatly increases the productivity of labor; the man who uses his wealth in this manner, therefore, is justly entitled to the increase of product secured thereby, and such increase is measured by the current rate of interest for safe investments.

The return secured by the use of capital varies greatly with the degree of industrial skill of the management selected and maintained in power by the financially responsible owners. Skillful owners and managers secure a much larger return on capital invested than the current rate of interest. Their skill creates this larger return under competitive conditions. This increase of return is measured by the profit, and owners and managers are justly entitled to the profit and loss of the management. Without this compensation skillful management would be but seldom obtained. To deprive skillful owners and managers of the profit they secure, under competitive conditions, would require the entire abrogation of present methods of selecting men for responsible control of all industrial enterprises. To abrogate profit, by limiting the return on individual enterprises to a fixed rate on the capital actually invested, would remove all inducement to efficient management. It would require such detailed supervision of all enterprises by the public as to amount practically to public management.

Extremely variable profits according to degree of skill of management are absolutely essential to efficient private ownership and management of industrial enterprises, and it is certain that any regulation attempting to enforce moderate uniform profits will lead to utter inefficiency of management and ultimately to the abandonment of regulation or of private ownership, and the introduction of public ownership and management. The control of all the important acts of the directorate must be placed with those who must bear the consequences of good or bad management. With uniform assured profits of individual enterprises the public bears these consequences, and must have full control and the appointment and discharge of the managers. The State cannot give to stockholders control and assured dividends; it can and should, however, do all in its power to protect them from being robbed by their directors and managers, by enforcing publicity of accounts, securing minority interests, and enabling them to form a correct opinion of the ability and honesty of their directors and managers. Mr. Mayer.

A reliable directory of presidents, directors, and general managers, giving the rate of return secured by stockholders in the enterprises with which they were connected, would do much to secure the rapid advance of good and the removal of bad managers and directors from influential positions.

Monopolies.—A clear idea of the nature of natural monopolies is next in importance in order to define the problem.

A natural monopoly arises when the advantages secured by production on a large scale, as compared with production on a small scale, become so important that the existence of a single producer for a product or service on a given market offers great economic advantages over that of several producers.

Transportation of freight on a large scale can be furnished by a railroad at one-twentieth the cost of transportation on a small scale by wagons on public highways. For local traffic, therefore, railroads are natural monopolies. For through traffic, where the amount much exceeds the capacity of a double-track road, two or more competitors are economical, but the number of competitors for securing the lowest cost of production is so small that agreements to maintain prices and to stop competition become practicable and extremely profitable to all the railroads concerned. Competition, therefore, disappears as a regulator of prices, and public regulation is necessary to secure justice.

The disappearance of competition is due to the cheapness of large-scale production. If there is no public regulation of prices, the capital invested in natural monopolies often brings much larger returns than that invested in competitive enterprises. This generally leads to extensive stock watering, by promoters and reorganizers, the profits of which often go to them and not to the investors. If such stock watering were prevented by the honesty of the management, or by publicity of accounts and public supervision of capitalization, and its limitation to actual investment, it would lead, without regulation of the prices of the products or services furnished, either to excessive dividends or to uneconomical management or both. The excessive profits obtained by production on a large scale, without competition, are not the result of the special skill or industry of the management; they go with competition to the public, and should be made to go there by the regulation of monopolies where competition disappears.

Mr. Mayer. This should be accomplished without robbing innocent investors and without destroying the efficiency of private management.

The present values of monopolistic enterprises are largely ascertainable from actual transactions of sale in the security markets, and are the results of estimates of the present value of future net earnings under the regulation which is now considered most probable and most in harmony with the present preponderant sense of justice. Where these values are excessive and unjust, the injustice is due to the past erroneous conduct of the community toward monopolistic corporations, and the community—not individuals selected by chance—should be made to bear the consequences of its past errors. To destroy or to revolutionize radically these values by the method of regulation adopted would be retroactive legislation, and, whatever legal maxims may be quoted to justify such a course, it would be robbing one class of investors for the benefit of others no more deserving. Such a course contradicts the fundamental principle of justice of compensation according to usefulness, and is thereby condemned. The new principle of regulation must be introduced with the least practicable injustice to past investors, and can only be applied strictly to all future investments. Justice, therefore, requires the method of valuation proposed in the paper.

To proceed to various special objections: The formula for ascertaining the average profit of an old enterprise is criticized as not being correct, because the rate of interest on the left side of the equation should be the legal or a fixed rate. To make the formula absolutely correct this rate of interest should be the actual one at the time and place considered. The rate used in the writer's formula is certainly closer to this than a legal rate generally uniform over a whole State and representing what was desired by the majority of the legislature, not what existed at the time of its enactment. The same critic proposes to omit interest altogether from the left side of the equation. This would introduce a serious error, would exaggerate the profits in enterprises which take a long time to develop, and would relatively under-estimate the profits in enterprises with a quick return. A regulation based on such omission of interest would be unworkable, because no capital could be secured for enterprises of slow development. There is no circular argument in the paper, and such a statement results from careless reading. Any regulation of prices requires the use of mathematics; to avoid this use, therefore, is impossible.

The purpose of the chapter entitled "Charging What the Traffic Will Bear" is to show in a general way why, and to what extent, monopoly prices are higher than competitive prices. The context shows that what is intended is to compare the prices under monopoly and competition with the same scale and method of production. The small change in quantity of demand which is considered in the argument will not produce so large a change in cost of production as to affect it materially. The writer believes that the inference drawn in this argument is substantially correct, though not required for the regulation proposed. Mr. Mayer.

A mere observation of the relevant facts shows that the unregulated monopoly prices are generally too high and should be reduced by just regulation. Monopoly prices are only lower than intelligent competitive prices when the monopolists do not pursue their interests intelligently, as, for example, when they engage in the production of a service or product, the degree of usefulness of which does not permit a price which would give an ordinary rate of profit; this case, however, is expressly excluded from the argument.

It is often economical to have only one producer for a local product. In this case, also, the competitive price—that is, the price giving to average ability average competitive profit—is lower than that charged by an intelligent monopoly pursuing only its own financial interest without fear of regulation.

The question has been asked: How would this regulation proceed if there is only one producer?

The excessive concentration of ownership and management of many disconnected enterprises is largely due to the effort to obtain monopoly prices. If competitive prices are enforced for monopolized products, it will not pay to concentrate ownership by buying up independent enterprises to a larger extent than is justified by the savings in cost of production thereby secured. The day of single ownership of whole industries will be thereby postponed indefinitely and need not concern us at present.

A frequent objection to the establishment and usefulness of such commissions, made privately, is based on experience with existing commissions, which is sometimes disappointing.

Sometimes, such commissions are not composed of men of sufficient ability, and they often work under impracticable laws. If these laws were strictly enforced, they would cause so serious a disturbance of the value of investments that the commissioners would be discharged. As a necessary result, little is done. Often, with otherwise reasonable laws, the funds available are entirely inadequate for establishing a strictly scientific regulation. One of the most successful commissions, though it has inadequate powers, is the Interstate Commerce Commission. It has been guided approximately by the principles advocated in the paper. It has regulated the rates without causing a regulation in security values. It has not attempted to secure a uniform rate of profit in different enterprises; it has endeavored to establish such rates as would enable the railroads to secure new capital for extensions and improvements in service, or such rates as would make the average profits on capital thus invested the same as in competitive enterprises.

Mr. Mayer. Railroads are for through traffic, which is largely competitive. With those rates which are the result of real competition the commission has interfered very little. One important result of competition, among enterprises in which the rate of profit largely depends on the amount of business done with a plant of large capacity, must here be mentioned. When there are several such independent competitors for a total amount of business far below their combined capacity, one of them, by reducing prices, may attract so much new business that his profits are increased by the reduction. The others, by their losses, may be induced to reduce prices still further in order to recover their customers. This may go on, with a drop in the value of the securities of the competitors, and may result in their consolidation, either before or after the bankruptcy of some of them. Such competition inevitably results in its elimination and in gross injustice, during the process, to many of the security holders. Among railroads, this is largely a phenomenon of the past, and is now prevented, either by gentlemen's agreements or by the just interference of the legislatures and the regulating commissions. Such unreasonably low rates do not give compensation according to usefulness; they are equally unjust with excessive rates, and, without regulation, inevitably result in unreasonably high rates thereafter, since they frighten away capital from enterprises in which they occur.

The laws establishing regulating commissions should demand such regulation of the prices of monopolized products as will yield equal average profits in competitive and monopolized industries at the same time and place. They should prescribe that the present value of each enterprise be determined by the same general principles as are used in the valuation of real estate. Future values for rate-making purposes should be determined from the present value plus the additional capital paid in by purchasers of newly issued stocks and bonds. The dividends and interest paid out should be considered as the profit of the owners. This regulation does not require any impracticable or very expensive valuation or determination of profits; it does no injustice to any class of present investors or to the consumer; it does not destroy the efficiency of private management, and will secure the same average profit for future investors in competitive and in monopolized enterprises. The only practicable alternative to such regulation is public management and ownership of all natural monopolies. If public ownership is established, similar commissions will be just as necessary as with private ownership. The efficiency of management of any industrial enterprise can only be determined by a scientific comparison of its costs of production with those of other enterprises producing similar services or goods. Such a comparison cannot be made by the voter or by any individual; it requires the systematic investigation of all similar enterprises of a large territory.

Competition, where it exists and where it can be economically preserved, automatically removes uneconomical, and increases the scale of operation of the most economical, managements. For public management this function must be performed by qualified commissions with equal relentlessness in order to secure a tolerable degree of efficiency and enterprise. The inefficient regulation of the prices of monopolized products has been largely responsible for the introduction of public ownership of industrial enterprises. If this regulation is of such a kind as to destroy the efficiency of private ownership, public ownership is the necessary result. Without harm to economy and enterprise, the public regulation of private enterprises may prescribe minimum wages, healthy conditions of work, compensation for accidents and sickness, old age pensions, and protection of women and children; it may prescribe the quality of the products or the services rendered; and it must prescribe the prices for monopolized products and services. Mr. Mayer.

The writer's aim has been to describe a method for the regulation of the prices of monopolized products and services which would make public ownership and management superfluous. Why should we fear public ownership and management? The public has but average intelligence. All improvements in production are first conceived by exceptional intelligence. The mind which originates an improvement needs a capitalist able to appreciate his invention and to introduce it into practice. If, by the prevailing industrial system, you assure to the capitalist the savings in cost of production obtained by the improvement until the new method has become common, then the inventor of a real improvement has a reasonable chance to get it introduced. With public ownership, the inventor of an expensive improvement must, for securing its introduction, first convince a large number of mediocre men who will each gain little by its adoption. Public departments, therefore, are unduly conservative, and progress is slower than with private ownership.

A board of directors composed of large stockholders is more likely to watch closely the economy of management and to select the chief executive officers with the exclusive aim of finding the best man, than a commission appointed by a mayor or elected by the voters. The chief executive officers of a stock company know that their efficiency will govern their tenure of office, and all the employees will be selected, retained, advanced, and discharged mostly by efficiency, while, thus far, considerations other than efficiency often have more influence in public than in private enterprises.

An inefficient private enterprise working under either natural competition or with prices regulated by an efficient commission will, at present, be more rapidly put out of business than an inefficient public department. Some better means than now available, however, are

Mr. Mayer. necessary for making a reliable comparison of the efficiency of private and public enterprises and for correcting erroneous opinions and consequent mistaken action in choosing either public regulation or public ownership. Thus far, competent public commissions are the only bodies that can efficiently perform this work.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

TRANSACTIONS

Paper No. 1226

ROAD CONSTRUCTION AND MAINTENANCE.

An Informal Discussion Presented at the Meetings of January 19th and 20th, 1912.

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(1) DRAINAGE AND FOUNDATIONS.

By Messrs. James Owen, Paul D. Sargent, Theodor S. Oxholm, F. C. Pillsbury, J. A. Johnston, Clifford Richardson, W. W. Crosby, Arthur H. Blanchard, Samuel Whinery, H. P. Willis, and Will P. Blair.

Mr. Owen. James Owen, M. Am. Soc. C. E.—The practice of road foundation is, to a large extent, crystallized, and it would seem that all that is necessary is to get an idea of what is fairly good. On the question of drainage, it may be recalled that, at one of the National Road meetings, the Minister or Commissioner of Roads, of Ontario, laid down three fundamental principles of road construction: First, drainage; second, drainage; and third, drainage. He was asked afterward what he would do if he lived in Arizona, where there is no rainfall. From this, it can be reasonably understood that drainage is: first, a matter of climate; second, a matter of topography; and third, a matter of soil. In discussing these subjects, it is best to consider the question of drainage *per se*.

There is no doubt that in 90% of the country there is sufficient rainfall to require the consideration of drainage, and that where there is lack of rainfall there are not many roads; consequently, drainage is a factor in road construction.

The problem can be divided into two: the removal of surface water and the removal of underground water. The assumption that the average macadam surface is water-proof at all times is not maintained by facts, and cannot be relied on as a basis for practical construction. It is possible, with a clean, unbroken surface, consolidated by constant travel into a homogeneous mass, that after a dry spell the first rainfall would be shed; but, after a continuous soakage, all broken stone becomes somewhat loosened and this makes the matter of drainage a bad proposition.

Water on highways can be shed in three different ways: First, by the crown in construction; second, by the grade after the crown is constructed; and third, by a discharge into some natural watercourse.

The question of crowning has been rather a matter of controversy, and the speaker's practice varies according to location, grade, and amount of travel, and also according to the character of travel. The question of trolley tracks on roads has also to be considered. On a country highway with a single line of travel, the best practice has always been to build the crown about 2 in. higher than is ultimately desired, as the work is green, to a certain extent, and, if allowance is not made for the first settlement, there is a likelihood of a flat surface in the center where water will collect.

It has been the speaker's practice to increase the crown gradually until, on a new 16-ft. road, it is 5 in. This will be compacted to about 3 or 3½ in., which is good average practice. In suburban towns, or on roads where the travel is divided or heavy, the necessity for so great a crown is not apparent. Where there is a trolley road, the crown is practically eliminated by the track in the center, and from it a slope has to be built to the gutter. That slope is, of necessity, reduced to a minimum, and the speaker has found that, in repairing roads along trolley tracks, an artificial crown of about 1 in. or 1½ in. will finally wear down to a normal slope sufficient to shed water. Mr. Owen.

In considering crowning, there is always the question of the grade of the road. As a rule, it must be understood that the steeper the grade the greater the crown. If there is a 10% grade, with an ordinary flat crown, the tendency of the water in a heavy rainfall is to follow the grade rather than the crown, or, at least, to make a curve in its final discharge; but in such cases it is always better to make a rather higher crown.

There is great objection to the high crown in thickly populated districts, due to the sliding motion of the vehicles, and also to the extra strain on the lower part of the wheels. Consequently, it is desirable to eliminate the crown as much as possible in consideration of this excessive strain.

When there is a bituminous surface, or even an oily surface, a flatter crown can be used than with ordinary macadam. On country roads the shedding of the water to its final outlet is generally done in the gutter. It is a better practice, and a matter of economy, even on such roads, to provide the drain pipes or sewer pipes along each side with inlets at intervals of about 400 ft., so as to gather the water and prevent a great rush.

Springs in the roadbed have to be treated according to the special conditions. The speaker wishes to emphasize particularly the fact that sometimes an undue amount of money is expended and care taken to provide for what, during construction, seemed to be a spring in the bed of a road. If there is a cut of 12 or 15 ft. in a gravel pan, it generally contains a spring, and the usual procedure is to make elaborate efforts to remove the water permanently. The speaker has found, however, after considerable experience, that the first rush of water is merely a lowering of the water-level to the new grade, and that frequently within a year or two such springs disappear entirely. Consequently, the money spent to take care of them was practically wasted. Therefore, the topography of a locality where there are springs should be considered, and judgment should be used in determining whether a spring is permanent or whether it is merely due to changing conditions in constructing the road.

Mr. Owen. In regard to grades, the speaker has always insisted that a flat road should never be built. In some localities it is almost impossible to accomplish this; but, where it is possible—and in ninety-nine cases out of a hundred it is—it is better to build the road so as to shed the water longitudinally rather than continuously sideways. The ordinary level macadam road may be rounding in its first construction, but the travel will wear off that rounding and there will be a depression in the travel line, where water will gather, soak into the road, and make a bad rut. In a long stretch of level country, it is sometimes difficult to undulate the road from point to point with a minimum fall of $\frac{5}{10}$ in. in 100 ft., but it is better practice than to have a level surface.

As to the question of limiting grades, the speaker's standard for a road is between 1 and 2 per cent. The former gives a very efficacious result. Less than 1% causes a retardation of the natural flow of the water, and such roads require more slope to conduct it to the gutter. More than 2% and up to 4% is a good traveling grade, and has been the speaker's standard for such purposes.

Another question enters here, which, while not related to the grade itself, is worthy of consideration. The first proposition was that on traveled roads, where there is a height of a certain number of feet to be overcome in a certain distance, a certain grade should be adopted. It has been the accepted practice for years to limit the grade to the lowest possible through rate. As a matter of fact, the question of time is a factor as much as that of load. If there is an 8% grade, the horse will have to walk up that grade with his load; but if it is cut into two grades, say of 2% for a section, with the remainder steeper, the horse can trot on the low grade, gaining time on that part of the road, and can walk just as fast up the steep grade as up that of 6%, which is an important matter where time is a consideration.

This same rule would apply to automobiles, and it seems fair to presume that with automobile travel, and also with auto-truck travel, there should not be so much insistence on low grades. A great deal of money has been spent in cutting down hills, and making deep cuts and fills in order to produce a low minimum grade. It makes very little difference to the average automobile, except as a matter of time and some shifting of gears, whether it goes up a 2% or an 8% grade. Consequently, this question has to be decided from the standpoint of the amount of travel and the character of the traffic.

One little point in the question of drainage is the problem of handling quicksands. In one case, in the speaker's experience, a road went through a very bad bed of quicksand. An open trough of wood was constructed with battens on the top and open at the bottom. This was laid on the quicksand, and the men stood on it and dug out

Mr. Owen.

the sand. As the trough gradually settled into place, it was filled with stone, and provision was made for an outlet for the water at the end. That piece of road has never stirred since it was laid thirty years ago.

The frost line, of course, is a vital question in drainage, as it is in all road construction. It relates more particularly to foundations, but the speaker wishes to emphasize the fact that in the consideration of all road construction, the question of temperature, where there is no frost in the ground and no obstruction to the underground flow of water, is very important; but where the frost is 4 or 5 ft. deep, breaking up the homogeneity of the road and coming out at the surface, with, say, 1 ft. of melted earth on the top and 2 ft. of hard frozen ground below, it makes the most undesirable proposition that can be considered, and these conditions will obtain almost all over the northern part of the United States. Here, naturally, rises the question of foundations. For a number of years, the speaker has been rather a lonely man in road construction, and has persistently and constantly advocated foundations. He has built very few macadam roads, and is very glad of it. He has built some, and has then taken them up and rebuilt them with foundations, and to-day, after a series of years spent in road construction and maintenance, is perfectly satisfied that, finally, whatever little extra cost was incurred originally in building foundations is counterbalanced by the smaller maintenance expense.

In two communities, in the same locality, with about the same population, and the same travel, a series of roads was built. One community built its roads with foundations, and the other thought it could get along and spread its money a little farther by putting down macadam. The final result confirmed the speaker's views, as the macadam roads broke up in a few years and necessitated a large expenditure to put them in proper shape. Some telford roads, built 6, 7, 8, 9, and 10 years ago, have stood up without a break, and, in one case, where a road was built of telford, it did not receive a dollar's worth of repairs in 19 years, and it was a heavily traveled road.

The speaker has adopted a principle which he has found to be fairly successful, namely, to make the thickness of the pavement according to the grade. The following data were established as a basis: Less than 1%, a 10-in. pavement; between 1 and 4%, an 8-in. pavement; and greater than 4%, a 6-in. macadam; and it would seem that such a principle might be taken as a governing one in considering the case of foundations for road construction.

The telford pavement has, or should have, if the construction is fairly good, a system of drainage of its own. Such a pavement is supposed to be in reality a drain to the bottom, after which the water has to be carried away to the lowest point.

Mr. Owen. In the question of foundations for future work, another element is intruding, which will require careful consideration, namely, the auto-truck. This auto-truck will take a load of, say, 5 tons, and weighs 5 tons itself, that is, 10 tons on four wheels, and if a road is constructed of 6-in. macadam, and is allowed to wear down to 3 in., and in some cases 2 in., the inevitable result will be that the auto-truck, or even any other heavy load, will break through, and the road is gone.

The question of cost is material. In the speaker's district, there is plenty of stone, and telford roads are built as cheaply as macadam roads within the same locality; but any frost-proof stone can be used in foundations, and the speaker has laid many miles of sandstone foundation and obtained good results from the roads. The great trouble at one time was the desire to spread the money over as long a distance as was possible—to obtain quick results—and this idea continued for a long time, and led to less care, from a purely engineering standpoint, than if the future had been considered. To-day that issue is not regarded as important. The public mind at large has wakened up to the necessity of good roads. It has been educated. It is easier now to get the amount of money necessary to make good roads, and if the practice can be crystallized, the United States will have as good a system of roads as any in the world.

Mr. Sargent. Paul D. Sargent, M. Am. Soc. C. E. (by letter).—The writer can heartily agree with all that has been said as to the necessity of thorough drainage and, as nearly as possible, a perfect foundation on which to lay any kind of a road surface. There is one kind of foundation work with which engineers sometimes have to deal, namely, that for a roadbed across a swamp or bog.

The writer, while State Highway Commissioner of Maine, constructed two sections of improved road over bogs which were very quaky and unstable. In each case the work was really widening an old road. In one section the center line of the new road coincided with that of the old one; in the other, all the widening was on one side of the original road. In both cases the bearing power of the bog was improved by the construction of a brush mattress under the new fill. For this work, boughs of pine, spruce, fir, or hemlock, were used, care being taken to exclude those having stems more than 2 in. in diameter. These boughs were laid shingle fashion in courses, first crosswise and then lengthwise of the road, and were four courses deep, that is, two transverse and two longitudinal layers, the total depth being about 16 in. On the boughs was placed a 2-ft. fill of gravel. One of these roads was built in 1908 and the other in 1910, and, to the writer's knowledge, not the slightest settlement of the foundation has occurred. Both bogs were so bad that it was impossible to drive a horse across them, and even a man jumping would shake them for a radius of 50 ft. Mr. Sargent.

This method of building a foundation was also used on one section of new county road, constructed under the jurisdiction of a board of county commissioners, where the bog was so bad that, even after the mattress was laid, it was necessary to unhitch the horses and draw the wagons out on the mattress with a tackle and fall before they could be dumped. After this road had been built about two years, reports indicated that it was in perfect condition, and that large loads were being carried over it.

In work of this kind, it is absolutely necessary that the turf or sod be not cut. If it is, and much of a load is superimposed, settlement will almost certainly occur. The writer has never tried laying a macadam road on one of these roadbeds, but has no doubt that, after the roadbed has been in use for a few years, a macadam surface could be laid on it, if a light roller was used for consolidating the crushed stone.

Theodor S. Oxholm, M. Am. Soc. C. E. (by letter).—During the past 14 years the writer has had charge of the construction of many miles of first-class water-bound macadam roads in the Borough of Richmond, City of New York. Previous to that, some good roads had been constructed by the county, with telford foundations. When repairs were made to these latter roads, it was found that the upper or wearing surface, composed of the smaller sizes of stone, was badly worn where it came in contact with the telford; this appeared to indicate that the wearing surface was being worn, both at the top by vehicles, and at the bottom by the impact on the heavy telford stone, thereby causing an increased expense for maintenance. Mr. Oxholm.

It has always been the writer's theory that, in any kind of macadam road construction, telford or field stone should only be used for drainage purposes; that on dry and well-drained soil, the ordinary macadam road, composed of stone ranging in size from 2½ in. to screenings and dust, is suitable for all classes of travel in country districts, where such roads are mostly built. The resiliency of the soil and the pavement itself (provided it is built of sufficient thickness, and under proper specifications), should protect the road from serious damage by the heaviest trucks.

It has been found in this Borough that large sums, formerly spent for unnecessary drainage, could be saved by constructing the road, in most cases, where the conditions appear to be favorable, without any special provisions for drainage other than well-cleared ditches at the sides, having suitable outlets at the foot of grades, and by noting frost heaves or other damaged sections during the winter and spring, and, in these places, which are comparatively few, building blind drains of stone from the center of the road diagonally to the ditches or culverts.

It has also been found that well-built tile drains, or flat stone

Mr. Oxholm. drains, have a tendency to fill up in a few years and become practically useless, so that where such drains are actually necessary, much more care should be taken in their construction.

Mr. Pillsbury. F. C. Pillsbury, M. Am. Soc. C. E.—There is no doubt about the necessity for adequate drainage on all roads, whatever the surface construction may be. Drainage and foundation problems, however, are varied, and their solution depends so much on natural conditions above and below the surface, as well as on variations in climate, that set rules, formulas, or plans should be prescribed carefully, as they would not be equally effective under different conditions. Frequently, there appears to have been an unnecessary expenditure for drainage and foundations by the use of so-called telford and stone V-drains. The speaker has frequently noticed places where such foundations have been used with apparent lack of judgment; places where these types of foundations were not necessary at all, or where some other materials, such as gravel, cinders, slag, etc., might have been used, at a much lower expense. He has also seen stone foundations used where they could not give good results unless laid on sand or sandy gravel. The speaker's experience has been, that for surfaces which are not subjected to extremely heavy loads, such as would require granite blocks or similar pavements, a sandy gravel provides a better foundation than telford or V-drains, under any natural conditions. Without adequate foundations and proper drainage, even the best surfaces fail. In fact, many failures which have been attributed to other causes have been due to this.

Mr. Johnston. J. A. Johnston, M. Am. Soc. C. E.—State roads have been built in Massachusetts for 17 years. The speaker has been connected with the work from the beginning, and for the past 15 years has been a Division Engineer, with jurisdiction over construction and maintenance in about one-fifth of the State.

In beginning its work, the Commission constructed some rather elaborate systems of trench drains and telford paving for foundations. Where such work was done there were no failures, but the cost was excessive. Such foundations were not built everywhere, but, on the contrary, the policy was to build them only where they were thought to be absolutely necessary, and to omit them in case of doubt. In view of the expense, the speaker believes that this theory was correct, and especially because at first greater reliance was placed on blind drains than has since been found to have been justified. Of course, it was expected, if the need developed, to build such additional drains and foundations as were required.

It has been the speaker's practice to locate carefully every frost break and soft spot which shows on the road in the spring, and to remedy such places as promptly as possible. It has been difficult at

Mr. Johnston.

times to persuade some of the past members of the Commission of the necessity for such work. Some of them have believed that it was a mistake to disturb the crust of a road.

To avoid breaking up this crust, many experiments have been tried. On one section of macadam road, which was so muddy in the spring that the crust had the consistency of porridge, from 6 to 8 in. of broken stone were placed on what had originally been 6 in. of macadam. This improved the conditions for one season, but in the second year they were as bad as before applying the remedy. This section was at the foot of a hill and on a fill about 2 or 3 ft. above meadow land. A blind drain, 2 ft. wide and 3 ft. deep, was then built across the upper end of the fill to cut off the ground-water from the hill, but it had very little effect. Similar cross-drains were then constructed at about 100-ft. intervals, but the road continued to be muddy. After that the macadam was stripped off and a foundation, 12 ft. wide, 18 in. deep in the center, and 6 in. deep on the sides, was built with field stones not exceeding 10 in. in their largest dimension. The old broken stone was then screened and replaced, but so much of it had been churned into the mud, that though from 12 to 14 in. had been placed on this road, scarcely enough was recovered to cover the foundation. Since this work was done, 5 years have elapsed, and there has been no further trouble.

Miles of ground-water drains have been built which the speaker now believes to be practically useless, for it is his opinion that under many conditions the capillary action of the soil, intensified by the puddling or tamping action of the traffic passing over a road surface, nullifies, to a large extent, the supposed effect of the drain in lowering the water-table, and this is regardless of the fact that there may be a water-proof top on the road. The speaker has built blind drains entirely around a section of road, 15 ft. square, and with proper outlets, but with no appreciable effect on the enclosed section. He does not absolutely condemn these drains, for there are soils which can be drained, and there are places where an excess of water must be taken care of, but there are many instances where more money has been spent on drains than would be required to build an absolutely unyielding foundation under the entire road.

Under most conditions, a 10-ft. foundation of stone, even 6 in. deep, which will seldom cost more than 25 cents per lin. ft. of road, is of immeasurably greater benefit than a single drain which will cost more money. To hold up a road surface by strengthening the foundation may not sound as scientific as by the more indirect method of drainage, but it is far more successful. By a stone foundation is not meant the hit-or-miss dumping of stone into mud holes, nor, at the same time, is it essential to hit each stone on three sides with a

Mr. Johnston. hammer and wedge it in place as telford paving. The dirt surface of the sub-grade should be smoothed and properly graded to an outlet for the water; the stone should be placed to grade, with the smaller stones on top to close the crevices, and the whole should be well rolled. It is not always necessary to have a great depth of stone, it should be varied according to conditions, being, in some places, 18 in., and in others only 6 in. It is by no means essential that the foundation should extend the full width of the macadam. On most of the roads in the speaker's division the foundations are only 10 ft. wide, while the macadam is 15 ft. In the spring, when the frost is coming out, the average width of travel is rarely more than 10 ft., and, though the eye can readily detect the line of the edge of the foundation, there have been no breaks and no trouble from this practice.

To use more than 4 in. of broken stone for macadam is a mistake, for anything below this depth is merely foundation, and for this, field stone at $1 per cu. yd. is much better than crushed stone which will cost from $2.50 to $3 per cu. yd. On very soft soils it may be advisable to use a layer of gravel under the foundation, but a light layer of field stone over the gravel will give a far better and more substantial road than the gravel alone. In 17 years of experience, the speaker never saw a serious failure over a stone foundation which had been properly laid.

Not long ago, a road official, who had been recently appointed, told the speaker that he intended to put a telford base under all his roads. Another official, also newly appointed, stated that he did not suppose any foundation was needed under 6 in. of macadam. Unfortunately, the speaker could not agree with either of them. In Massachusetts there are many miles of road where the macadam is laid on the natural soil, with excellent results, and many more which would not have lasted one year without a foundation.

While engineers may be justified in experimenting with ordinary macadam, which can be taken up and relaid at a cost of about 14 cents per sq. yd., it is criminal negligence to take the same chances with expensive pavements, though all have seen deplorable examples of this fault.

Mr. Richardson. CLIFFORD RICHARDSON, M. AM. SOC. C. E. (by letter).—There is nothing truer than the old adage that a road, like a house, should have a dry cellar, a firm foundation, and a tight roof. Without these characteristics a road cannot be of the highest quality, especially if the soil on which it is built carries ground-water at a depth of less than 3 ft. below its surface. There are, of course, many soils of a gravelly, sandy, or similar character, which are self-draining and do not require particular attention. The defects in most country highways which are built on clay soils, or those which do not drain them-

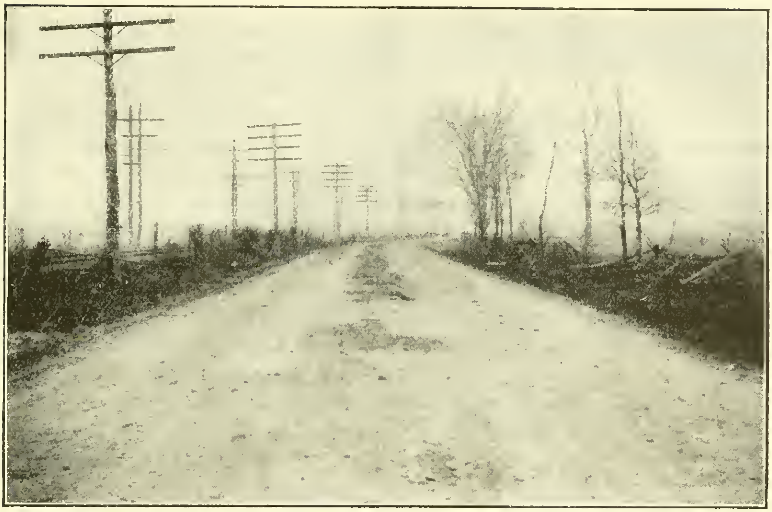

Fig. 1.—Waterford Road, Water-Bound Macadam Thrown by Frost, March 30th, 1910.

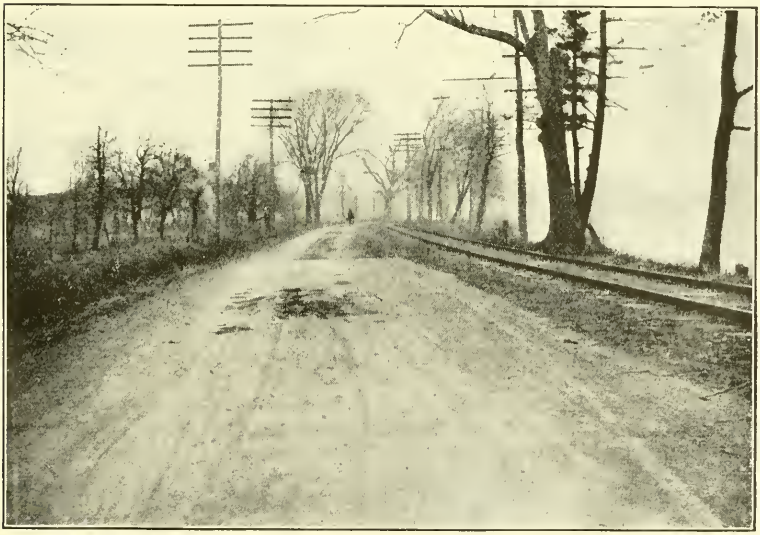

Fig. 2.—Waterford Road, Laid in 1909, Showing Junction of Genasco Asphalt Macadam and Water-Bound Macadam, Both Thrown by Frost March 30th, 1910.

selves, can be attributed to lack of drainage. This is true, not only of the roads built in the United States, but also to a very great extent of English roads, as will appear from statements of English highway engineers to which the writer will refer. Mr. Richardson.

There is probably no highway engineer who does not recognize the importance of drainage, but it is an astonishing fact that, nevertheless, very few roads are properly drained. This is generally neglected because a proper study has not been made of the character of the subsoil on which the road is built and, perhaps more so, because of the additional expense it involves. When some of our public officials state that a good broken stone highway of a modern type can be built for less than $6 000 a mile, the engineer hesitates to increase the cost to a point which will lay him open to the serious criticism of extravagance, by introducing proper drainage.

Mr. Frank D. Lyon, Second Deputy Commissioner of the New York Highway Department in 1910, stated, in a paper on the "Location and Drainage of Highways":

"Among the road builders of to-day good drainage is recognized as one of the most important considerations, whether the roads in question be of earth or those with a covering metal. No one subject involved in the construction, repair or maintenance of an earth, gravel or macadam highway is of as much importance as that of drainage."

Although he recognizes the importance of drainage, it is safe to say that most of the State highways in New York are not provided with a means which will keep their foundations free from water, and dry. Figs. 1 and 2 reveal the difficulties encountered after building a piece of bituminous broken stone road on an old water-bound, broken stone surface, the crust of which was placed directly on clay. During the first spring after it had been opened to travel, when the frost began to come out of the ground, it will be seen that the bituminous surface at the center was thrown up and out of place. The situation is quite the same in most of the States, although in Massachusetts it has received somewhat more careful attention.

Mr. W. A. McLean, Provincial Engineer of Highways, of Ontario, Canada, recognizes the same situation in his country as that occurring in the United States. In a paper on the subject, he says:

"Roads in Canada to-day are bad for the same reasons that they were bad in England a century ago, before the time of Macadam. They are drainless quagmires, swallowing the stone and gravel placed on them. Townships commonly spread stone on their roads and speak of them as being 'macadamized.' To macadamize our roads means, in the first instance, that we must thoroughly drain them by surface and under-drainage. The essential principle of a macadamized road is drainage. This was the principle advanced and introduced by Macadam, and it is the one so commonly neglected throughout Canada to-day."

Mr. Richardson.

In building highways, the Canadian engineers, like those in the United States, while recognizing the importance of drainage, have been and are, it would seem, unable to carry into practice the ideas which they recognize as essential in road construction.

The situation is equally unsatisfactory in England, as can be seen from the statements of a prominent English engineer, Mr. R. O. Wynne-Roberts, who, in considering the question of drainage, says:

"The first consideration in connection with all roads is that of sufficient drainage, but unfortunately there are hundreds of miles of highways without satisfactory means of draining off the subsoil water and afterwards of conveying it away; the same remark often applies even to the surface water. In the case of water bound roads—and those constitute the principal portion in this country—the presence of water or excessive moisture keeps the subsoil in a sodden condition, thereby so reducing its sustaining powers as to make it unable to bear the concentrated loads often imposed, with the result that the metal coating is deformed, disintegrated, and in some measure pressed into the soft subsoil. At the same time the displaced subsoil oozes upwards, causing the roads to be softer than before, muddy in winter, dusty in summer, expensive to maintain, and giving rise to dissatisfaction to the road authorities who maintain, and to the public who use the highway in question. Economy of maintenance of a public road is largely governed by the condition and character of the subsoil. It is highly desirable that its weight carrying capacity should be preserved and improved by efficient drainage."

Mr. G. H. Jack, County Surveyor of Herefordshire, England, in considering the subject of the improvement of the roads in his county, said in his Fourth Annual Report (1911):

"The trunk roads have not in many cases a crust thicker than 4 inches and this crust rests directly on the clay subsoil. It is possible that with efficient under-drainage and impervious surface this clay may be kept sufficiently dry all the year round, and if so, then the many troubles arising from the yielding clay may be surmounted. I do not, however, consider this a certainty with a water-bound crust, however well the under-drainage is carried out. The very fact of the permeability of the surface would cause the underlying clay to yield under heavy weights in wet weather. The principal function of the drains would be to get rid of the underground water and not so much the water which falls on the surface. This can be quickly disposed of through the grips and ditches if the surface is water-proof. I do not by this suggest that under-drainage would not vastly improve the condition of the water-logged lengths of roads made under our present system. On the contrary, I have cases in mind where I know the result of deep draining would be most beneficial, and would certainly prevent the roads entirely giving way as they did in many cases last December."

These statements are sufficient indication of the general neglect in all countries of the proper drainage of highways. It seems to be a purely administrative and not an engineering defect. The engineer knows the importance of drainage, but the administrator and the taxpayer fail to recognize its necessity, and are unwilling to meet the expense which it involves. Mr. Richardson.

It is unnecessary to consider here the question of how proper drainage should be carried out. The proper methods are very generally recognized, and almost any highway engineer can provide for such drainage, if the necessity in any particular case is realized and the money is forthcoming, by putting in pipe drains along the sides of the road at a depth of at least 3 ft., or deeper if necessary, to remove the ground-water, or, where large quantities of field stone are available, by blind drains in the center of the roadway at a sufficient depth. These types of drains have been discussed so thoroughly in highway literature that it would seem to be a waste of time to go into the subject at this time. Surface drainage has been given more careful attention everywhere than that of the subsoil and foundation.

Finally, a statement of Mr. Vernon M. Pierce, of the Office of Public Roads, made in a paper presented to the Second International Road Congress at Brussels, may be cited. He stated that road-builders are learning more and more from experience that, as a rule, it is cheaper in the end, and more satisfactory, to drain roads than to lay expensive foundations. It is certainly true that there can be no object in constructing an expensive foundation unless the subsoil on which it is laid is dry.

The foundation is as important an element in road construction as drainage, but a foundation is useless in a location where the subsoil carries or holds ground-water, unless proper drainage is provided. Proper foundations for roads have been neglected the world over to the same extent as drainage. It is an astonishing fact that an engineer who would not, for a moment, think of constructing a building without a foundation, will expend large sums of money for a road surface without providing any means for its support. The writer will cite some evidence in connection with this situation from statements of prominent highway engineers at home and abroad.

Mr. Jack, of Herefordshire, has something to say in his Third Annual Report (1910) in regard to his roads, which would apply equally well to many locations in the United States. He says:

"The surface coatings of our roads are so thin and the subsoil so unstable (mostly clay) that a continuously wet autumn in some cases had ruinous effect. The passage of heavy engines over such roads caused more damage than can be readily described; it is not so much the damage to the surface as the squeezing of the subsoil which causes so much trouble. Over many miles the sides are weak and unmetalled, and consequently the passage of heavy weights tends to, and, in fact does, flatten the crown of the road, and destroys effectual

Mr. Richardson. drainage. Complaints have been made as to the roads being too flat in cross-fall; in many cases there is cause for complaint, but it is largely due to the compressing of the subsoil. It is certainly not due to wilful negligence on the part of the surveyors in one of the most elementary principles. Instructions have been given unceasingly for the contour to be neither too flat nor too round, but of sufficient convexity to throw off the water and sufficiently flat to induce the traffic to use the full width of the road instead of adhering to the practice of keeping in one track—a practice which greatly adds to the difficulties of road surveyors. It does not appear to be generally followed out that there should be more curvature on hills than on level roads, and consequently in many places I have noticed the water follows the middle of the road instead of taking to the water-tables."

The same situation will be found in many parts of the United States. Mr. Jack adds:

"I have been surprised to see how some of our roads have deteriorated in a single year under motor and engine traffic, and I feel sure that if we do not now take the work seriously in hand it will be a matter of remaking rather than maintenance. The time has arrived when we are forced to view our work in a very different light. It has become a labour much more arduous and calling for all the skill the surveyors are capable of, and that coupled with their constant attention. I have great faith in what can be accomplished by personal concern, and indeed, I am sure that if the human element is lacking in energy and interest we cannot hope for good roads, however much the county council spend. There appears to be a doubt in some quarters as to the extent of damage to road due to motors. I think this can be set at rest by reviewing the cost of county roads during the last ten years.

"In 2	counties	the	cost	has	increased	over 100	per cent.
" 2	"	"	"	"	"	81 to 100	"
" 12	"	"	"	"	"	41 to 80	"
" 8	"	"	"	"	"	21 to 40	"
" 13	"	"	"	"	"	0 to 20	"

"I am glad to say that Herefordshire is one of the thirteen counties showing the least increase. At the same time our Ross to Hay Road, which is one most used by motors, has increased in cost during the past ten years by 100 per cent. The paramount difficulty in Herefordshire is to be found in the unstable subsoil and want of pitched foundations."

Mr. Jack adds, in his Fourth Annual Report:

"The heavy cost of maintenance of the roads used by engines and heavy motors, not only in Herefordshire, but in very many other counties, arises from the lack of a firm foundation. The construction of strong roads means, of course, the expenditure of large sums of money. It is an undoubted fact that roads, like any other structure, will give way if they are not laid on solid ground. In many cases building up the surface to the required thickness (11 in.) will suffice, but in others nothing short of a proper foundation will prevent incessant expenditure and a poor road into the bargain. It is not likely that a water-bound road (which means a pervious surface) will carry traction engines when the road crust is not more than 4 in. in thickness, resting on clay, which becomes of the consistency of dough when wet. Even if deep subsoil drains are laid in such roads the clay is of such a retentive nature as to make the passage of water to the drains a very slow process." Mr. Richardson.

Mr. E. P. Hooley, County Surveyor of Nottinghamshire, England, in speaking at a meeting of the Institution of Municipal and County Engineers, in August, 1911, said:

"There was hardly an inch anywhere. In the County of Notts there were just the foundations which had been put in during the last few years. If they had to start and put in the foundations of the roads in the rural districts, putting in the necessary 6 in. of foundation, plus the top material, they were going to such an expense that no council would keep it. If they could put on 3 or 4 inches of material on the top and get as good a result, they would still be considered foolish if they put in expensive foundations."

As in England, much of the difficulty with broken stone roads in the United States has been, and is, due to the subsidence into the soil of the crust of the road. In many cases the soil has not been properly prepared to receive this course, and in others the nature of the soil has been such that it would be impossible to prepare it for the purpose without very careful drainage, which has been omitted almost everywhere, or by the construction of a suitable foundation of the telford type, or one of hydraulic concrete. A concrete foundation, of course, will be more costly than one of broken stone, but a foundation of this type, 4 in. thick, will be stronger than a loose one of $2\frac{1}{2}$ in. broken stone, 6 in. thick, and the difference in cost will not be a serious consideration, because the much smaller quantity of stone used in the 4-in. foundation will largely make up for the cost of the cement and sand in the concrete. A concrete foundation, when once laid, will give an asset which can be counted on for all time, while a broken stone foundation is necessarily of a more or less temporary nature, owing to its displacement under travel and its disappearance into the soil which supports it. An effort should be made, at least in the case of building those highways which are to carry heavy travel, to construct them with a concrete foundation, which will last for all time, and on which a wearing surface can be constructed easily and economically at such intervals as may be necessary. It is evident that, as engineers are constructing for the main arteries of heavy travel, they are throwing away large quantities of money by failing to provide adequate support for the wearing surface. The result is much the same as if expensive buildings were erected without adequate foundations to support them. This question of adequate foundations is one which should receive

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the most attention to-day. It is one which, if neglected, will do more to hamper the development of good roads than anything else, as great disappointments must arise within a few years, when it is discovered that the fine wearing surfaces which are now being constructed are so inadequately supported that they have given no reasonable return for the cost incurred in building them, and have rapidly gone to pieces.

Mr. Wynne-Roberts, who has already been mentioned, in taking up the question of foundations for roads, has made the following statement in regard to the situation in England:

"The foundation is an essential feature of a good road, but it has not been adequately provided on a large proportion of our highways; consequently the thin crust of metal which sufficed to carry the traffic in former days is found to fail under the present-day conditions. Great expenditure of public money has lately been incurred in strengthening such roads. Some have been reconstructed, many more have been provided with thicker coating of macadam, without remedying the evils of defective drainage and foundation. Where the subsoil is hard and dry the absence of solid pitched foundations has not a serious effect on the quality of the road, but where the subsoil is wet and yielding, true economy can only be effected by thorough reconstruction, or by employing binders—which are unaffected by dampness—with the extra layer of macadam. The county surveyor of Wilts, in his last annual report, states that where the subsoil is rock or chalk, if well covered, it will be found that the cost of upkeep of the road will be from 30 to 40 per cent. less than that with the same traffic where the subsoil is clay, green-sand and silt. Telford, Macadam, Tresaquet, and other prominent road engineers, advocated stone foundations, but even they found it was not possible, probably owing to financial reasons, to adhere rigorously to their ideals or dogmas; and if it was the case when the highways were first being constructed on scientific lines, when labor was cheap, traffic comparatively light, except on some mail roads, it will doubtless be admitted that to reconstruct these roads will tax the monetary resources of the numerous road authorities, and also of the Road Board. Still, in view of the ever increasing expenditure on these roads, due to the recent rapid development of traffic, satisfaction and economy will only be attained by judicious application of modern methods of construction and maintenance. If it is possible to render a road which is simply a thin crust of metal lying in a bed of clay, and requiring annual repairs, into one which will stand the same traffic with practically no repairs for about three years, by the adoption of the modern systems, it should not require much argument to convince the road authorities that real economy is attainable at small cost."

Mr. J. Fred Hawkins, County Surveyor of Berkshire, has the following to say on the subject:

"In road making, as in house building, the foundation should be the first and chief consideration. In modern road construction the formation and foundation are, no doubt, considered above all things,

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but this was hardly the case in the days when most of our main roads were formed for coaching and other old-fashioned types of road traffic. It is often stated that most of our trunk roads have good foundations, and in some cases it is so, this chiefly in counties where the natural foundations are of rock or hard stone, or where granite is easily obtained, and has been used as road material for years. In counties like Berkshire, however, where there is a dearth of quarries, the chief material used in forming the road was Thames ballast and local gravel; the roads can be said to have no foundations at all, thus making it impossible to keep a good surface, every ton of hard stone rolled in being almost at once swallowed up. * * *

"What would the highway authorities of those days think could they see the traffic on the Bath road to-day? In spite of the extraordinary change in the traffic on our roads, the foundations still remain the same, and it is impossible to expect those roads which have gravel foundations, and the surface of which is water bound, to stand the abnormal weights of heavy motor and traction traffic. This traffic is permanent, and is bound to increase."

The statements which have been quoted are but a few of the many which have been made, which are at hand at the moment. The general opinion of British highway engineers is of the same character as those quoted.

The following is quoted from a recent editorial on the Second International Road Congress at Brussels in an English journal:*

"We have often drawn the attention of our readers to what we consider to be a well-established fact—viz., that the modern road problem is a problem of foundation rather than one of surface. * * * In regard to drainage, roads should be designed so as to prevent infiltration of water into their surfaces, which should be made as impermeable as possible."

The situation brought about by the lack of foundations, which has always been a serious one, has been intensified by the increased weight of the traffic which is using the roads at the present time, and it is much the same in the United States as in Europe, at least on the main arteries of travel. The eventual solution of the problem will be the construction of concrete foundations for roads of this type, especially in cases where the traffic is as great as shown by the traffic censuses, as on many of the residence and less used streets of cities, where a concrete foundation is always considered necessary. As a matter of fact, as the writer has shown elsewhere, the most economical method of treating main highways which are subjected to heavy motor travel, will be by paving them with some of the various surfaces (on a concrete foundation) which are able to resist the continuous traffic to which they are subjected in cities. At first sight, this would seem to be an extremely expensive proposition, and, if it were followed, the

* *The Surveyor and Municipal and County Engineer*, August 12th, 1910.

Mr. Richardson. mileage of roads which could be constructed with the money available, at the present time, would be much reduced, but eventually the situation would be largely improved.

At the Second International Road Congress at Brussels, the following conclusions were arrived at, in discussing the subject of road foundations:

Foundation.—1. The strength of road foundations should be increased in proportion as the supporting power of the ground decreases. The foundation should have more body and resistance, the more it is exposed to internal deterioration and external wear.

2. In the choice of the system of foundation for both stone block pavements and metaled roads, due consideration should be given to the condition of the subsoils, with regard to the possibility of their drainage, to their geological nature, and to the nature of the materials of the locality. In order to determine the thickness and the extent of the foundations, the pressure per unit area should be made compatible with the carrying resistance of the soils, observed under the most unfavorable conditions.

Drainage.—3. In soils where preliminary drainage is required before the construction, the general methods should be applied to the whole or to a part of the roadbed and to the bed of the metal, if necessary.

4. The cross and longitudinal sections of roads and those of side-gutters should be established so as to facilitate the flow of water, and to prevent infiltration into road surfaces, which should be made as impermeable as possible. The evaporation of superficial dampness should be encouraged by every means.

5. The works for the foundation and for drainage should be carried out simply and economically, and by using the materials of the locality as far as possible.

Of course, these conclusions mean but little, are very colorless, and merely show that drainage and foundations were considered of importance by those drawing them.

The subject, however, cannot be too frequently brought to the attention of highway engineers and tax-payers in America, where the situation is plainly as bad as it is shown to be in England by the quotations from the statements of the authorities on road building of that country. It is to be hoped that American engineers will give the matter serious attention, for, unless these features of road construction and the economics of the problem, that is to say, the manner in which the building of roads is financed, are considered, there will be an enormous revulsion of feeling toward the construction of good roads when it is discovered that those which are now being built give no adequate return for the money expended.

W. W. Crosby, M. Am. Soc. C. E.—With the development of traffic along the lines of heavy trucking, the proper construction of road foundations is more important than ever. The surfacing with bituminous material of many roads near Baltimore, Md., has invited and encouraged their use by heavy motor trucks for the purpose of bringing farm products to market. Mr. Crosby

Until recently, macadam, 4, 6, or 8 in. thick after rolling, was perhaps a standard surfacing, and proved satisfactory where there was a proper sub-grade. Even now there appears to be a general tendency among engineers, as well as contractors, to consider a sub-grade sufficient, provided it is possible to build a surfacing on it. In the speaker's opinion, however, this is not always a correct assumption.

That part of the foundation which comes next to the surfacing and is ordinarily known as the sub-grade, is one of the most important, and warrants more attention than it usually receives. If this sub-grade is not properly prepared, it may churn up into the stone when this is applied, with the result that the actual thickness of the clean stone layer is materially reduced. On bad soils, it is sometimes difficult to get the surface of the sub-grade into first-class shape. For instance, in clays, especially in those which contain considerable mica, the sub-grade, during the process of rolling, will flake after becoming partly compacted, these flakes appearing both ahead of, and behind, the roller. This condition, in the case of sub-grades in cuts, may be caused by too much rolling, but it frequently occurs on fills which the speaker is absolutely certain were not rolled too much. This flaky condition may be overcome satisfactorily by spreading a layer of sand, cinders, or stone dust, as may be convenient, on top of the sub-grade, to a thickness of from 2 to 6 in., and then compacting this layer with the roller, so as to secure an entirely satisfactory surface on which the macadam can be placed without danger of its being mixed with the sub-grade material and thus losing any portion of its effective depth.

Still another method, which has been found to be quite successful, is to use for the first course of macadam, crusher-run stone containing all the fine material. Such stone, placed on a poor sub-grade previously put in the best possible condition, is then thoroughly harrowed. The effect of the harrowing is to deposit the fine material in the bottom of the stone layer and produce apparently about the same effect as the application of a layer of sand or screenings placed by itself on the sub-grade as previously described. (It is assumed, of course, that proper drainage has been secured in all cases.)

In some localities, it may be possible to build roads, under favorable conditions, for a figure as low as $2 000 per mile, but the speaker believes it to be the duty of engineers generally to resist the pressure

Mr. Crosby. of any uneducated, uninformed, and incorrect opinion which holds that the average cost of satisfactory roads should be anywhere near that figure nowadays, and to insist on such expenditures for first cost as will result in economy in the long run. The first cost is not a true measure of expense. Maintenance costs must be taken into account. Unfortunately, figures have not been collected with proper accuracy, nor for a sufficient length of time, to show exactly the cost of maintenance, in the long run, and what the cost of improper construction may be. If such figures were available, the speaker believes that there would be more popular support for building roads with greater first cost, and that there would be far less public demand for the cheaper roads which are generally unsatisfactory.

Mr. Blanchard. ARTHUR H. BLANCHARD, M. AM. SOC. C. E.—It is the opinion of many engineers that the heavy commercial traffic to which the main county roads of England are to-day subjected will be characteristic of the traffic on our trunk highways outside of built-up areas within the next 5 years. English engineers are now confronted with the problem of providing adequate foundations for many miles of macadam roads in use. If this situation is to avoided in the United States, it is self-evident that foundations should be of such character as to be able to carry the traffic to which the road is likely to be subjected in the near future at least. It appears to the speaker that the standard of 4-in. broken stone foundation courses on certain subsoils for trunk highways will have to be replaced by 6 to 10-in. foundation courses of broken stone, or a concrete foundation, if heavy commercial motor traffic is to be carried without deleterious results.

During the past year the speaker has adopted a maximum crown of $\frac{1}{2}$ in. per ft. for macadam roads which are to remain as ordinary water-bound macadam or are to be finished later with a bituminous surface. Under certain circumstances as low as $\frac{3}{8}$ in. per ft. has been advised. The same recommendations have been made relative to the crown of bituminous pavements, but in cases where a smooth surface finish was assured, or the bituminous pavement was completed by the application of a seal coat, a maximum of $\frac{3}{8}$ in. and a minimum of $\frac{1}{4}$ in. per ft. was prescribed.

Mr. Whinery. SAMUEL WHINERY, M. AM. SOC. C. E.—The problem of foundations involves, of course, that of rolling. It is a fact, which some may not have observed, that the amount of rolling which any foundation will bear depends on the character of the material. Plastic clays in excavations, for instance, if they are not disturbed, are about as thoroughly compacted by Nature as they can be. Whenever a heavy roller is put on such a surface, and it is rolled too much, a condition of plasticity is produced. The speaker thinks that very frequently clay foundations are over-rolled, and that the remedy is to stop the rolling the moment one notices a tendency of the surface to wave, either behind or before the roller. In his experience in preparing street foundations in excavation, a 4-ton roller has often given very much better results than a 10-ton roller. Mr. Whinery.

Of course, it is not necessary to state that an adequate foundation is as essential for roads as for other structures. In his discussion Mr. Richardson has given great prominence to concrete foundations. The speaker thinks that it is quite impossible to prepare any standard and unalterable specification for road foundations. The rational practice in road construction, as well as in street paving, is to study the requirements of each particular case, and then to adapt the specifications and construction to those requirements. It has been the practice of most engineers, in dealing with concrete street foundations, to assume that practically one standard of thickness and quality of concrete may be used throughout a whole city. This is a great mistake, and much money is often wasted in that way. In the first place, there are many locations which do not require a very strong foundation, but where one of concrete can be used more advantageously than any other kind. It is not necessary, however, to use the best concrete, for a very small proportion of cement may give the requisite strength. In places where cement is costly, a much leaner concrete can often be used, and where the other materials are comparatively cheap, it may be found more economical to use a cheaper foundation of such lean concrete, because the strength of a beam increases with the square of its depth.

H. P. Willis, Assoc. M. Am. Soc. C. E.—The speaker believes that the subsoil must be considered as the real foundation of the road, and that, before the road problem is solved, everybody will come to this conclusion. The principal agent in the destruction of the sustaining power of a soil is moisture, most of which gets into the road by capillarity from the sides. The ditch fails to remedy this condition, because it exposes a greater surface through which the moisture can pass in this way. All road builders agree that the surface constructed above the foundation should be impervious to water. That being so, the question is how to shut the water off from the bottom and sides in the most economical way. If a ditch is dug along the side of the road at the edge of the macadam, the natural plane is broken. If an ordinary soil pipe, about 4 in. in diameter, is laid in this ditch and covered simply by tamping back the excavated soil, whatever it may be, the conditions have been changed in such a manner as to benefit at least the roadway, as far as concerns the moisture getting in by capillary attraction, because the water follows the broken plane. Water trying to reach the ground-water level, acts as a vertical force, and if that force can be made more than the Mr. Willis.

Mr. Willis. horizontal force which tends to take it under the road, it will be of great benefit in keeping out the water. If the material taken from the ditch is a clay, and if it is puddled when filled back over the pipe, a wall which will be impervious to water will be provided on the outside of the road. The water which tends to seep into the road from beyond the pipe will meet this impervious wall, follow it down, and flow off through the pipe. Mr. Lyon, of the New York State Highway Department, has tried this method with success, and it is the cheapest treatment known to the speaker, costing about 4 cents per ft. Another method of obtaining the same result is to cover the pipe with broken stone, which answers the same purpose, and probably more effectually, but is more expensive. The New York State Highway Department is now working on a scheme whereby a trench will be dug next to the macadam, and nothing will be put in that trench but a series of vertical plates made either from clay of the same kind as the soil pipe, from asphalt and clay mixed, or, in fact, from anything which can resist the disintegration which will take place in the ground. The water will meet that plane, follow it down to the pipe, and run off.

Mr. Blair. WILL P. BLAIR, ESQ.*—The object of trenching is to keep the ground dry under the wearing surface. Instead of laying the pipe sub-drains at the sides, parallel to or along the center line of the road, the speaker advocates laying them at right angles to the center line. It is his practice to lay ordinary 4-in. vitrified clay pipe at intervals of from 8 to 10 ft. along the road in this manner. The pipes slope both ways from the center to the side ditches. They are laid below the frost line, and are back-filled with the material excavated in making the trench. This method has been found to be both cheap and effective.

* Secretary, National Paving Brick Manufacturers' Association.

(2) FILLERS FOR BRICK AND BLOCK PAVEMENTS.

By Messrs. George W. Tillson, Theodor S. Oxholm, E. A. Kingsley, Samuel Whinery, L. P. Sibley, D. E. McComb, W. A. Howell, W. W. Crosby, Will P. Blair, H. B. Pullar, Arthur H. Blanchard, and George W. Tillson.

George W. Tillson, M. Am. Soc. C. E.—The materials now being used for block pavements are stone, brick, wood, and asphalt, and the matter of filling the joints is of great importance. The object of the filling is to make the surface water-proof, prevent undue wear of the individual blocks, and preserve the continuity of the pavement. Brick and stone produce a noisy pavement, and, for these types, the filler should tend to reduce this noise, if possible. At present, specifications call for joints in such pavements to be filled with sand, tar and gravel, coal-tar pitch, or other bituminous material, and cement grout. Mr. Tillson.

Naturally, when the first crude pavements were laid, a cheap and available filler was used. This led directly to the use of sand, which gave fairly good satisfaction for those times, when it was not deemed necessary that the pavement should be water-proof. When an attempt was made to improve the pavement, however, the blocks were made of a more regular shape and laid on a concrete foundation. From a sanitary standpoint, the pavement was required to be water-proof, and a filler other than sand was sought. The material decided upon was gravel, ranging in diameter from ¼ to ½ in., the interstices being filled with coal-tar pitch. This method was used in London in 1869, in Manchester previous to that time, and in New York City by the Dock Department in 1881. When brick pavements were first introduced, sand was used in the joints, followed later by cement grout and coal-tar pitch. The spaces between the blocks of the old cedar pavements, so much in vogue in the Central West between 1880 and 1890, were filled generally with sand, but sometimes with tar and gravel, the latter materials being used when the blocks were laid on concrete. The joints between the blocks of the modern wood pavement have been filled with sand, cement grout, or with coal-tar pitch, or some other bituminous material. The filler for asphalt blocks has almost invariably been sand, though sometimes, in relaying old blocks, the joints have been filled with asphalt.

Stone Pavements.—The first blocks used for pavements were of stone, and the joints were filled with sand, as has been stated; but such a filling should only be used for a temporary pavement. A sand filler does not preserve the edges of the blocks from undue wear, and consequently they soon round off, presenting a cobble-stone effect to travel. Water soaks through the joints during rain, dampening the foundation.

Mr. Tillson. Urine from horses also saturates the sand at times, giving off unwholesome odors when it evaporates. When concrete foundations were first introduced for stone pavements, it was realized that sand joints would not do. The tar and gravel joints previously referred to were then adopted, the idea being that the gravel would keep the blocks in place, and the tar, filling the spaces between the gravel, would render the entire joint water-proof. Theoretically, this was undoubtedly true, but, practically, there was often enough fine gravel to prevent the tar from filling the entire joint; or the tar itself, as well as the gravel, would become cold, thus preventing a free flow, the result, in many cases, being a joint which was far from water-tight. Then, after some time, the upper part of the joint would wear, sometimes to the depth of nearly 1 in., so that the edges of the blocks would wear off, producing what is known as the turtle-back effect, which is almost always seen in old pavements of that character. To use gravel of such a size as would permit the free flow of the tar made it necessary to have wide joints, which was also conducive to the undue wear of the blocks, often making a rough and unsatisfactory surface, even when the blocks themselves were fairly good.

Two improvements over the tar and gravel joint have been used: a straight coal-tar pitch or some other bituminous compound, and Portland cement grout. To make the pitch joint satisfactory, it is necessary to lay the blocks close, even stone to stone, and dressed so that there will be no excess spaces to be filled. With narrow joints, the pitch will hold the blocks in a stable position and also prevent the direct transmission of the impact of traffic from one block to another. A stone pavement is bound to be noisy, but one laid in the foregoing manner is thought to be as free as possible from this defect. The blocks being laid close prevents undue wear on the edges, such as seen with tar and gravel or sand joints. A notable instance of a pavement of this character, laid during 1911, can be seen on Fourth Avenue, Borough of Manhattan, New York City. Here, special requirements were made for both the pitch and the blocks, those for the latter being probably more stringent than for any work of a similar character in the United States. The requirements were lived up to, and the result has been a highly satisfactory pavement.

Worcester, Mass., was probably the first city to make use of the cement grout joint. The spaces between the blocks were first filled with stone screenings and then poured full of the grout. The result was very satisfactory. As the blocks were gradually made better and therefore laid more closely, the screenings were left out entirely, and grout alone was used. There are advantages and disadvantages in a joint of this kind. It makes a smooth and even pavement, and one that is easily cleaned. It presents little resistance to traffic, and brings the wear of traffic vertically on the top of the blocks, so that

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the pavement is very durable. As the joints are filled full of the grout, which wears down evenly with the surface, the pavement itself becomes smooth and slippery under wear, both on account of the smooth surfaces of the blocks and because of the lack of joint depressions to give foothold to horses. It is also extremely necessary that the grout should become thoroughly hard before any traffic is allowed on the pavement, otherwise the blocks will be loosened and the permanent set prevented, so that the filling becomes little, if any, better than sand. Pavements thus laid become practically continuous, giving off a metallic noise under traffic. The hardened grout, being a good conductor of sound, transmits the noise readily from block to block. Where the grade is flat, however, and the street is kept free from traffic until the grout has thoroughly set, the pavement will be very satisfactory. It is the smoothest form of stone pavement that has been laid in the United States. Most specifications provide that the street shall be closed to traffic for a period of 7 days after the grout has been poured, in order to give the latter time to set. A pavement of this kind presents great obstacles to plumbers or subway men. It is exceedingly hard to open, many of the blocks breaking rather than separating from the next adjacent ones, thus causing a great deal of waste. It is difficult, too, to repave an opening properly, as traffic forces itself on the freshly grouted blocks, thus breaking the bond.

The advantages of a pavement of this type are that it is smoother, more durable, and more easily kept in shape than any other; the disadvantages are that it is more slippery, noisier, and harder to repair. The specifications for cement joints in various cities are much alike, but those of Newark, N. J., are perhaps the most elaborate. They alone, however, provide that the grout shall be kept on the surface of the blocks even with the highest part, and that the mixing shall be varied so as to make the grout vary in hardness according to the hardness of the particular stone used. The following is quoted from the Newark specifications:

"After the pavement has been brought to a uniform surface, Portland cement grout shall be poured into the joints until it appears on the surface. The grout shall be broomed into the joints, if necessary, to fill the same, and the operation must be continued as the grout settles until the joints are thoroughly filled flush with the surface of the blocks, immediately after which the entire pavement shall be broomed to a smooth surface, sufficient grout being applied to bring said surface even with the highest part of any of the blocks. The blocks shall be wet by sprinkling immediately before applying the grout if the condition of the atmosphere requires this precaution to be taken.

"The cement grout shall be composed of one or more measures of the best quality of freshly burned Portland cement to not to exceed one measure of clean, sharp sand. The mixture to be employed for

Mr. Tillson. any piece of work shall depend upon the hardness of the granite used, and this mixture shall be ascertained by sufficient experiments to give a grade which will be of such a strength as to wear down uniformly with the granite used. In the mixing of the cement and sand, clear water shall be used, to give the proper consistency.

"The grout must be mixed for this purpose in a box about four (4) feet eight (8) inches long, thirty (30) inches wide, and fourteen (14) inches deep, resting on legs of different lengths, so that the mixture will readily flow to one corner of the box, the bottom of which shall be six (6) inches above the pavement. The mixture must be removed from this box to the street surface with scoop shovels, all the while being stirred in the box as the same is being emptied. One such box to be provided for each ten feet of width of roadway. The work of filling should thus be carried forward in line until an advance of fifteen to twenty yards has been laid, when the same force and appliances shall be used to regrout the same space in a like manner, excepting that the proportion of the mixture shall be two parts Portland cement to one part sand. To avoid the possibility of causing the grout to become too thick at any point, there should be a man with a sprinkling can, the head perforated with small holes, slightly sprinkling the surface ahead of the sweepers. To insure the penetration of the grout into the joints of the pavement there shall be used, in addition to the brooms, a squeegee scraper, 15 to 18 inches in length, on the last application of the grout.

"Within one-half to three-quarters of an hour after the last coat has been applied and the grout between the joints has fully subsided, and the initial set is taking place, the whole surface must be slightly sprinkled and all surplus mixture left on the top swept into the joints, bringing them up flush and full. After the grouting is done and a sufficient time for hardening has elapsed, so that the coating of sand will not absorb any moisture from the cement mixture, one-half inch of sand shall be spread over the whole surface, and in case the work is subjected to a hot summer's sun, an occasional sprinkling sufficient to dampen the sand should be followed for two or three days.

"After the grouting is completed the streets shall be kept closed and no carting or traffic allowed until at least seven (7) days have elapsed on any portion of the street grouted, and the face of the pavement shall be kept moist if the condition of the weather requires this precaution. The contractor shall erect sufficient and well constructed barricades, and furnish watchmen at all times, if the same shall be necessary, to insure that this precaution in regard to the prevention of traffic or carting is complied with. Should the bond between the blocks become broken for any reason, the joints at such places shall be cleaned out, even if it is necessary to take up and relay the blocks, and such parts so taken up and relaid shall be regrouted and rebarricaded."

With the improved stone specifications, most cities are using the cement joint at the present time.

Wood Pavements.—The first joint filling in modern wooden pavements was sand. Afterward, Portland cement grout and bituminous fillers were used. The speaker has always used sand. Wood blocks are so regular in form that they lie closely together in the pavement, and need a filler only to keep them in place. It may be said that with a sand filler the pavement will not be water-proof, but experience seems to demonstrate that the blocks, under traffic, soon mat together, making a surface which is practically continuous. The speaker recently examined a pavement of this character which had been subjected to light traffic for some 7 or 8 years, during which time it had been perfectly satisfactory. The sand should be fine, and thoroughly dry when applied, so that the joints will be entirely filled. Should oil at any time exude from the blocks, the sand will assist in absorbing it. Mr. Tillson.

Where a cement grout is used, it is made of equal parts of fine sand and the best Portland cement, carefully mixed, and swept into the joints until they are completely filled. The pavement is then covered with sand and the grout should be allowed to set for at least 7 days before the pavement is used. If the blocks are disturbed before the grout has set, the filling becomes of no more value than sand, and, as far as its absorptive properties are concerned, is even of less value.

Coal-tar pitch, asphalt, and special bituminous fillers are also used quite extensively by different cities, the idea being to make the pavement water-proof as well as to provide for some slight expansion of the blocks. Where such fillers have been used and excessive bleeding has occurred, much of it has been attributed to the bituminous filler.

In addition to filling the joints between the blocks, most specifications provide for expansion joints along the curb, and, in some cases, for joints across the street at intervals of about 50 ft. These joints are generally 1 in. in width, and are filled with either pitch, asphalt, or some similar material. Where the blocks are thoroughly treated with proper water-proofing, the cross-joints do not seem to be necessary.

There seems to be considerable difference in the practice of cities regarding the joint filler for wood pavements. For instance, the specifications of St. Louis, the Boroughs of Manhattan and Brooklyn, New York City, and those adopted by the American Society of Municipal Improvements call for sand; those of Detroit, for sand or paving cement obtained from the direct distillation of coal-tar, or any other approved composition; those of Indianapolis for an asphalt filler prepared from such asphalt and flux, if the latter is needed, as will conform to the specifications for asphalt paving cement; it must not be brittle at 32° Fahr., nor flow at 120° Fahr., it must adhere firmly to the blocks, and be sufficiently pliable to permit expansion and contraction; and those of Newark require a filler made of 1 part Portland cement and 2 parts sand.

The specifications of Westminster, England, require the following: All pavements shall be laid so as to leave as little space as possible at the sides and ends of the blocks, and, on completion, a mixture of

Mr. Tillson. boiling pitch and creosote oil, in approved proportions, shall be poured over the whole surface, be well forced into the joints, and be scraped off with wooden or rubber squeegees, the joints being thoroughly filled. The pavement shall then be finished with fine sand and cement grout in equal proportions, brushed over, and have a top dressing of approved gravel which will pass a ½-in. mesh, and be free from sand.

The specifications of the Organization for Standardizing Paving Specifications permit pitch, asphalt residuum, or sand, the latter being recommended for heavy traffic.

Brick Pavements.—As already stated, sand was used for a joint filler in early brick pavements. This, however, did not give general satisfaction, especially when the bricks were not laid on concrete, as it did not protect the joints from wearing. In many places, however, where the traffic was not heavy, sand has been used successfully. The following is taken from a letter received recently by the speaker from a prominent official of a large city in the Southwest:

"We have been using sand as a filler for the joints, for the reason that the bitumen filler is chilled before it gets to a sufficient depth between the blocks to serve its purpose. Where a dry sand has been used, the results have been entirely satisfactory. This applies not only to streets with heavy traffic but to those where the traffic is light. However, I have insisted on there being some stable header along the pavement; otherwise I would recommend the bitumen filler instead of sand.

"With the first brick pavements laid in this city, sand was used as a filler, and the results were satisfactory. The sand-filled pavements, after twelve years of service, look better than many of the grout-filled pavements."

When it was learned that the sand filler did not give good general satisfaction, both Portland cement grout and a bituminous filler of some kind were used. The advantages of the bituminous filler are that it decreases the noise and also takes up the expansion of the bricks. It does not protect the edges of the bricks from wear, however, and they round off under traffic, making the pavement rough. Some cities, however, now use the bituminous filler to a great extent.

The Portland cement grout filler is desirable, because, by filling the joints between the bricks even with the tops, the joints become part of the pavement, and, if properly made, are of practically equal strength with the brick, so that the pavement as a whole wears smoothly. In practice, however, two faults have developed with the cement grout filler: as it makes the pavement continuous, it expands, sometimes to such an extent that the pavement blows up with a loud report when the expansion is longitudinal, and sometimes, when it is transverse, the curb is displaced or broken. Often, when the expansion is slight, it apparently raises the pavement up from the sand cushion, so that the traffic causes a disagreeable rumbling noise.

Mr. Tillson.

There was a case of this kind in Brooklyn where the noise was so great that the pavement, although in good condition, was taken up and asphalt was laid in its place. Afterward, however, another brick pavement was laid in the same manner, but with more care, and there was no trouble whatever, although no expansion joints were used. By the observance of more care in the construction, the rumbling spoken of has been practically overcome, but expansion joints along the curb have been considered necessary. These joints are filled with coal-tar pitch or some other bituminous material.

Another defect in the early grout-filled pavements was that they often cracked and sometimes broke down, with the apparent failure of the grout in considerable portions of the street, while other portions remained intact. This occurs to some extent in grout-filled stone pavements. Its cause is uncertain; it is possibly the lifting of the pavement under expansion, as already mentioned, or possibly the unequal mixing of the cement and sand for the grout. The following is quoted from the letter previously referred to:

"I find that many of the brick pavements in the Western cities are disintegrating, due to the stresses caused by contraction and expansion, and not to what is ordinarily supposed to be wear and tear. From my experience I find that grout composed of one part Portland cement and three parts fine sand makes a very desirable filler for brick pavements, and one which I have been recommending for alley paving, and expect to include in the street paving specifications."

The practice to-day, as gathered from the specifications of leading cities, is to fill the joints in brick pavements with cement grout composed of equal parts of cement and sand, although Columbus, Ohio, has specifications for a coal-tar pitch and asphalt filler. The specifications proposed by both the Organization for Standardizing Paving Specifications and the American Society of Municipal Improvements recommend Portland cement grout, but give requirements for coal-tar pitch and asphalt.

It also seems to be the general practice to have at the curb an expansion joint ranging from 1 to 1½ in. in thickness, filled with asphalt or some other bituminous filler.

Asphalt Block Pavements.—The filling of joints in asphalt block pavements seems to be done almost invariably with sand, the blocks being of such hardness that, under traffic, they practically weld together in warm weather, and do not require a filler except to hold them in a stable position until such action takes place. Sometimes, in repairing old pavements, when the old blocks are relaid, an asphalt filler is used, as it seems to revivify the asphalt in the blocks, making the edges stronger and better able to resist the traffic. This, however, makes the pavement a little more slippery than a sand filler, and when the blocks are laid on a grade, to avoid slipperiness, the bituminous

Mr. Tillson. joint would not be desirable. On practically level streets, however, it has generally proved satisfactory.

In the foregoing notes the speaker has attempted to show the general practice at the present time as regards the filling of joints of block pavements of all kinds, with the reasons for using the fillers, and his opinion as to the best. In summarizing, therefore, he would say that:

As regards stone pavements, sand, tar and gravel, and cement grout are being used, the latter much more extensively than either of the others, especially where an improved form of pavement is desired. His opinion is that where a stone pavement is not laid on a heavy grade, and where the elimination of noise is not an important element, the cement joint will be the most satisfactory, provided the street can be kept closed long enough to allow the grout to set thoroughly. Where the elimination of noise is important, he would prefer the joint described in referring to the Fourth Avenue pavement in the Borough of Manhattan.

For wood block, he is thoroughly in favor of a sand joint, as he believes that with it the wood pavement will be water-proof, even under a moderate traffic, that it will provide for the expansion of the blocks to a certain extent, and is the cheapest material that can be used.

For brick pavement, he believes, without qualification, that a Portland cement grout filler, with an expansion joint along the curb filled with bituminous material, is the best.

For asphalt block pavement, he believes in the sand filler, as already described, for a new pavement, occasionally using asphaltic cement in repairing old pavements.

Mr. Oxholm. THEODOR S. OXHOLM, M. AM. SOC. C. E. (by letter).—The writer has had charge of laying a considerable yardage of vitrified brick pavement in the Borough of Richmond, City of New York, during the past ten years. A bituminous filler was used on much of this work. At first, it was composed of ordinary No. 4 coal-tar, but later a filler made from asphalt was used. The tar, while it adhered well to the bricks in summer, was apt to crack, powder, and blow away in winter. The asphaltic cement filler had a leathery appearance, and failed to adhere properly to the bricks, there being only a few months in mid-summer when a satisfactory piece of work could be had. In addition to this, it was noted that the filler wore away much faster than the bricks, thereby permitting their edges to spawl, thus materially increasing the roughness of the pavement in a few years.

While a pavement of this class is undoubtedly much less noisy in the first year or two than one in which the filler is cement grout, it is well settled that the increase in noise makes the pavement eventually more objectionable from this cause than one where the joints are filled with cement grout. The latter, in the writer's experience, has given a far superior piece of work, and the hollow sound sometimes noticed, due principally to lack of proper expansion joints along the curb, is less objectionable than the rumble of partly worn pavement with the asphalt cement filler. Mr. Oxholm.

Where traffic on a cement grout filled street is such that repairs to plumbers' cuts, etc., cannot be properly guarded for from 7 to 10 days, it would seem advisable to make repairs with tar or asphalt fillers.

Mr. Kingsley.

E. A. Kingsley, Assoc. M. Am. Soc. C. E.—No stone block pavements have been laid in Little Rock, Ark., in recent years, but the older pavements were laid in very much the same manner as described by Mr. Tillson. In the Southwest, creosote block pavements are laid almost entirely with a sand filler, the blocks being set comparatively close and the sand being put on and carefully brushed in. No expansion troubles result, and the oil in the blocks produces a practically water-proof surface. No asphalt blocks have been laid in this section of the United States.

Concerning brick pavements, the speaker's experience does not agree in every respect with Mr. Tillson's recommendations. There is no doubt that, where cement grout filler has been carefully put in, according to the specifications of the National Brick Manufacturers' Association, it has generally produced first-class results. It would seem, however, that there are reasons for not using it generally. The cement grout filler makes the pavement especially slippery during wet weather, and always slippery on hillsides, unless a special hillside brick is used to give an intentionally rough pavement. A bituminous filler has been found to be just as satisfactory for water-proofing a pavement as one of cement grout, and, if it is of first-class quality, there is no possibility of the pavement cracking. Again, unless there is provision for adequate expansion joints, a cement grouted street will expand at times sufficiently to crush some of the brick, and has even been known to explode. With a bituminous filler this is not possible.

The question of noise enters very largely into the use of brick pavements, and is one of the main objections to them. The bituminous filler acts as a cushion and reduces to a minimum the noise made by traffic over a brick street. The argument that a cement filled brick street is smoother than a bituminous filled street is not effective. In Little Rock, two years ago, a street was paved with brick, an asphalt filler being used. This street, under very heavy traffic, is as smooth to-day and as perfect as the best laid asphalt street in the city.

One of the most important considerations, especially in the smaller cities and rapidly growing towns, is the question of street repairs. Even the advocates of a cement grout filled pavement are the first to acknowledge that it is impossible to repair such a pavement

Mr. Kingsley. satisfactorily without destroying the bricks which are taken out and furnishing new ones. Consequently, repairing under manholes and over plumbers' ditches becomes a difficult and expensive proposition.

This same condition exists in regard to street-car tracks. It is exceedingly difficult to construct a street-car line so that, after a reasonable length of time, it will not have some loose joints. With a properly constructed bituminous filled street, it is not a difficult matter to remove the brick and rebuild the foundation, tightening up the loose joints and rebuilding the street with the brick taken out. Had the pavement been grouted with cement, it would have been impossible to remove the bricks without destroying most, if not all, of them; it would have been a much greater expense; and it would have been necessary, of course, to purchase new brick. Since the freight rate alone on paving brick runs from $6 to $8 per 1 000, the amount saved in repair work becomes a very strong argument in favor of the use of a bituminous filler, especially in the Southwest.

A fact sometimes overlooked is the quality of the filler. As much care and attention should be given to the quality of the asphalt which goes into a satisfactory paving filler as to that which goes into the asphalt street. The satisfactory filler must meet all the requirements for the different conditions of temperature, moisture, and traffic. It should be applied to the street with just as much care and attention as is given to the laying of an asphalt pavement, or the filling of a street with cement grout under the specifications of the National Paving Brick Manufacturers' Association.

The third reason for using a bituminous filler is the fact that the street can be opened to traffic immediately after the application of the filler. This cannot be done with a cement grout filler, as the street must be kept closed for a period of from 7 to 14 days before traffic can be safely permitted. This, of course, causes a little inconvenience for a short period, but is hardly a sufficient reason for advocating the use of another material, if cement grout were acknowledged to be the best.

Taking all things into consideration, and summing up the arguments advanced by the Paving Brick Manufacturers' Association, the speaker—as a user and not as a maker of brick for paving—cannot concede that a street filled with a good, soft filler is not as satisfactory as a noisy cement grouted street. In the Southwest it has been found that a brick street filled with a first-class asphalt filler is far more satisfactory to the property owners and residents than a cement grouted street. If the bricks are good enough to put into a street, so that they do not need a harder material to keep them from going to pieces, the life of the pavement will be as long when filled with asphalt as with cement grout.

Recently, a prominent Ohio brick manufacturer, who is not in favor of cement grout, because he manufactures a first-class brick, sent to a number of engineers in the central part of the United States, six questions regarding brick paving. In reply to the question: "What is your opinion of a good asphalt filler compared with cement?" about twenty engineers responded, and, with two or three exceptions, the opinion was in favor of asphalt. In only two or three instances were answers received favoring a cement filler, and, in some instances, excuses were given for so doing. In only two answers a preference was expressed for cement, without any qualifications. Mr. Kingsley.

Another of the questions was: "Do you have any cracks, breaking up of brick or cement, or arching in streets filled with cement?" In every instance where cement grout was used or favored the answers were in the affirmative, showing that with this filler troubles were always caused by cracking or arching. This series of questions was sent by this brick manufacturer without furnishing any information as to his opinions regarding fillers, but merely as a matter of information for himself. They are given as interesting data, and certainly they endorse the contentions of those who have fought so hard for the use of bituminous filler, and against which a few brickmakers have fought just as hard.

SAMUEL WHINERY, M. AM. SOC. C. E.—The speaker agrees substantially with Mr. Tillson's conclusions. He has been a strong advocate of cement grout filler for many years, and remains so. Mr. Whinery

The argument, that block pavements filled with grout are more difficult to repair, does not deserve great weight, unless one accepts the definition of a prominent member of the Engineer Corps of the Army, who, when asked what was the best pavement for city streets, replied, with some irony, that it was that pavement which could be torn up easiest and put back easiest. If this were the true definition of a good pavement, then, of course, the use of cement grout filler would be wrong. The fact that it costs a little more to make openings and repairs is of small importance when compared with the aggregate life and cost of maintaining the whole pavement over the street.

In the case of wooden block pavements, an exception should be made. The joints are purposely made so small, that it is quite impracticable to get any filler other than a fine sand into them; even where a grout filler is introduced with much care, the oil which exudes from the blocks has the effect, apparently, of disintegrating the thin sheets of mortar, and it becomes, in time, little better than plain sand. In the speaker's judgment, however, for all other classes of block pavement, under nearly all conditions, the grout filler is advisable.

L. P. SIBLEY, ESQ.*—There is an interesting development in the use of pitch filler for wood block pavement which is worthy of attention. With brick and granite block, a sand cushion is necessary to Mr. Sibley.

*Asst. Eastern Manager, Barrett Manufacturing Company.

Mr. Sibley. provide resiliency, but wood blocks themselves have sufficient resiliency. Three years ago the street railway company in St. Louis, in paving between its tracks with wood blocks, used a new method which eliminated the sand cushion entirely. The concrete base was put in as usual and surfaced with cement mortar to the grade of, and practically at a level with, the underside of the block. When the concrete and mortar had thoroughly set, and immediately before setting the blocks, the mortar was coated with hot pitch, and, as the blocks were set, a side and an end of each was coated with pitch by simply dipping the side and end into that already on the concrete at the point where the block was to be set. In setting the blocks, the coated sides and ends were placed against the uncoated sides and ends of those previously in place, thus positively insuring a thin layer of pitch to the full depth of every joint.

The advantages of this method, apparently, are: First, the concrete base is thoroughly water-proofed; second, the underside of the block is water-proofed, and as moisture enters the block through the end of the grain, and the surface soon becomes so dense that the moisture cannot enter from the top, the water-proofing of the underside largely protects against expansion, which is the most serious defect in wood block paving; third, it provides for any slight expansion which may take place in the blocks, by having pitch the full depth of every joint; and, fourth, it insures that all the pitch shall be below the surface of the pavement, where it serves its proper function, instead of a considerable part of it being on the surface, as is the case when the joints are poured or the pitch is flushed on, thus adding to the trouble if oil exudes from the blocks.

The speaker fears that some remarks with regard to the construction of brick pavements in Cleveland and Columbus, Ohio, may have created a wrong impression. It has been stated that cement grout filler is used exclusively in Cleveland. As a matter of fact, more than 5 000 000 lb. of pitch were used for filler in that city in 1911, and, based on a maximum of 15 lb. per sq. yd. for brick paving and 25 lb. per sq. yd. for granite block paving, this would provide for more than 200 000 sq. yd. of paving where grout was not used. In Columbus, where a pitch filler had been used almost exclusively up to two years ago, a grout filler was used to a considerable extent in 1911, but the latest information from that city is that this filler has not been satisfactory, and that, for the greater part of all the block paving in 1912, a soft filler will be used.

There may be grounds for a difference of opinion as to the results obtained with grout and soft fillers in pavements which do not have to be disturbed in order to make repairs to water pipes, sewers, water conduits, etc.; but where such openings are made in a grouted pavement, the original results cannot be obtained in the repair work, unless the repaired section is closed to traffic for from 7 to 10 days, otherwise the bond in the cement will be destroyed. Since it is impracticable to keep traffic from each repaired space for so long a time, a pitch filler was used in nearly all the repairs to brick pavement in Cleveland in 1911. In selecting a pavement, or any feature of it, it is foolhardy not to take into consideration the openings which must be made in practically all pavements during their probable life. Mr. Sibley.

D. E. McComb, M. Am. Soc. C. E.—The speaker desires to call attention to the difficulty of using grout filler successfully in pavements adjacent to street railways, as the operation of the cars prevents the proper setting of the grout joints near the rails. Under such conditions, in constructing granite and scoria block pavements, the speaker uses a soft filler for a width of ½ m. from the rail, and a grout filler for the remainder of the pavement, with an expansion joint of soft filler adjoining the curb. Mr. McComb.

Attention is also invited to the fact that it is more difficult to repair pavements with grout filler than those with soft filler; in addition to the more difficult removal of blocks, it is, in many cases, practically impossible to keep traffic away from the repaired pavement long enough to permit the cement to set properly.

W. A. Howell, M. Am. Soc. C. E.—In Newark, N. J., there are about 45 miles of brick pavements which have been constructed from time to time since 1895. The cement grout filler there has been entirely satisfactory. About 2 500 sq. yd. of pavement were laid 10 or 12 years ago with a soft filler which was not a success. For a number of years, owing to political influence and to the objection of a number of property owners to the noise, a sand filler was used, but it proved very unsatisfactory. At the present time, the grout filler, with 1½-in. expansion joints along the curbs, is being used. A number of years ago the speaker visited Kalamazoo, Mich., where both the grout and the soft filler were used. The experience there at that time seemed to be in favor of the soft filler. There may be climatic or other reasons for using a soft filler in the Middle West which do not obtain in the Eastern States. Mr. Howell.

Cities along the Great Lakes, such as Cleveland and Toledo, Ohio, and Erie, Pa., with very highly satisfactory lake sands at their command, should be able to get better results from cement grouting than is obtained in Newark. Although there are at least eight or ten of the best cements in the United States in this district, the local sand is not at all reliable, and even with the greatest care one can hardly expect to achieve the results attained in the Lake cities with only ordinary attention.

W. W. Crosby, M. Am. Soc. C. E.—A great many brick pavements have been laid in the vicinity of Baltimore, Md., during the last few Mr. Crosby.

Mr. Crosby.

years. The speaker has had charge of several hundred thousand yards of such work, and practically all of it has been built with a grout filler. He fully appreciates the difficulties encountered in using a cement filler with a brick pavement, but, from his own experience and from his observations of the experience of others in different localities, he thinks that the most satisfactory results are obtained with it, if the work is properly done. The speaker has in mind one street, which he sees nearly every day, which was laid 3 or 4 years ago with a soft filler. While its traffic is extremely light, because there are residences only on one side, the bricks are considerably chipped, and the pavement is quite noisy. It does not give a rumbling noise, but the sharper rattle which results from joints which are open at the surface. The specifications used in Baltimore for brick pavements are practically those of the Manufacturers' Association. At the present time, expansion joints filled with pitch are not used except along the curbs.

Mr. Blair.

WILL P. BLAIR, ESQ.*—It may be taken for granted that the use of brick and stone roads is only justified on the streets of cities and towns and on thoroughfares and excessively traveled country highways such as those connecting county seats and principal towns, those bearing the converging travel, and those passing through thickly settled communities or agricultural districts yielding heavy tonnage.

It is apparent, therefore, that certain definite qualities must be taken into account: Ease of traction, durability, sanitary qualities, comfort in use, economy in first cost, economy in cost of maintenance, and the beauty of the pavement itself.

It is the speaker's purpose to show to what extent and in what way the use of a cement filler in a brick pavement contributes to the realization of the qualities mentioned. In the consideration of the question, however, it must be understood that the cement filler must be proportioned and applied properly. It should be composed of one part of fine, sharp sand and one part of an approved brand of Portland cement.

By the use of a cement filler the important element of ease of traction is greatly assisted and a monolithic surface is formed. The bricks protect the thin portion or cement joint, thus insuring uniform wear on the whole surface. The slight unevenness of the bricks, which will obtain for the first few years, according to the amount of traffic, will prevent slipping and skidding, which otherwise might occur, owing to the glaze which is always present on every No. 1 paving brick. As this glazed film disappears in time, the roadway becomes smoother, the granular surface is exposed, and a non-slippery condition of the surface follows. The same degree of ease of traction is not found with any other form of pavement whatsoever,

* Secretary, National Paving Brick Manufacturers' Association.

and is never approached in the case of a brick or stone pavement constructed with any other filler. Mr. Blair.

The hardened cement filler is sufficiently tough to withstand the shock from impact without shattering. With the relief afforded by the uniform 2-in. compressed sand cushion which is a necessary adjunct in the transmission of the load from the monolithic wearing plate, the joint is not broken, the vibrations of the impact on the wearing plate are distributed without injury, and the load is not concentrated wholly on any individual brick. With the monolithic plate resting on the uniform cushion support, the cushion itself is not affected or disturbed, except to the minutest extent; whereas, in the use of the soft filler, a continuing maximum disturbance occurs. The bricks are subject to constant displacement. Their support cannot be uniformly maintained, hence the surface is divided into as many planes as there are bricks in the street.

Where soft fillers are used, the force, or the entire weight, comes on the single brick as the stroke or wheel comes in contact with it. Two paradoxical conditions follow the use of a cement filler: The street grows better as it grows older, and the smoother it wears, the less slippery it becomes. Of course, this does not hold good indefinitely, but it does hold good for an undetermined number of years. With the use of soft fillers, it is certain that chipping at the corners and edges of the bricks will immediately follow after the street is in use, and this leads to periodical settlements which are quite apparent. The bricks do not chip where the cement filler is used. When granite blocks wear in this manner it causes a smooth, rounded condition of the stones, which subjects the horses to most cruel and incessant short slips, thus impairing their value and shortening their lives.

The wear on cement-filled pavements is scarcely perceptible from year to year; it is slight and even. No waves or depressions are produced, hence the impact is always at a minimum, and, naturally, there is less wear and more comfort in use. From every conceivable angle the advantage is with the cement filler, both as to ease of traction and durability.

Accepted information from medical scientists imposes a duty of co-operation on the Engineering Profession, which cannot be ignored. The cement-filled street best fulfills the sanitary requirements. There can be no contamination of the soil under it, and the accumulation of offal and filth can be taken from the street with the least trouble, the least expense, and more completely than from any other form of pavement surface known. The flushing of streets with water, which is objectionable for some other forms of paving, is readily accomplished, and does not harm the cement-filled brick pavement.

The only condition remaining—that of noise—is more a dream than a reality. Noise from a cement-filled street is often due to other

Mr. Plair. avoidable causes. One thing is absolutely certain, the noise from grout-filled pavements is slight when compared with that from pavements in which the filler does not protect the edges of the bricks or stones from chipping or wearing into boulder-like surfaces. It happens sometimes that the noise from a grout-filled pavement is attributed to the filler itself, when it is caused by the fact that the sand cushion was not properly compressed and compacted prior to laying the brick on it. This leaves hollow spaces between the brick and the cushion and results in a rumbling noise. There is a Medina sandstone pavement in the southwest corner of the public square in Cleveland, Ohio, which has been built for about 17 years. It was constructed with a grout filler, and there never has been any complaint whatever by the business interests adjoining that pavement on account of noise. During this period it has been subjected to a traffic equal perhaps to that of any street in New York City, ranging from 14 000 to 18 000 vehicles every 24 hours, with all kinds of tonnage that usually pass over streets in the business centers of large cities. The stones are not rounded off, and a level monolith is maintained.

During the summer of 1911 the streets of Cleveland, Ohio, were subjected to a change of temperature more severe probably than in all the past history of the city. About three-quarters of the brick pavements in that city are grout filled, and, of all the 2 700 brick street intersections, only 27 were ruptured. In examining those intersections, it was found that in no case where a rupture had occurred was the expansion cushion ample enough. When the expansion was exhausted, there was necessarily a raise or rupture in the street due to lack of sufficient provision.

In Columbus, Ohio, up to two years ago, soft fillers were used in the construction of brick pavements. During the last two years, however, brick pavements with grout fillers have been laid, and are equal to the pavements of Cleveland. For 15 years the latter city had steadfastly adhered almost exclusively to the use of grout fillers, both for stone and brick pavements. The popularity of brick and stone pavements in Cleveland has been so great that it has extended into the country until now there are 400 miles of brick-paved country highways in Cuyahoga County, with 72 miles arranged for the coming year. In Grand Rapids, Mich., about 75% of the brick streets are constructed with a cement grout filler. Although two or three streets there have the soft filler, the speaker does not know of any having been used during the last eight or ten years. In Kalamazoo, Mich., there is perhaps more soft filler used than in many other places.

The cost of a cement-filled pavement is not at all excessive. The actual cost of cement fillers is not as great as any of the so-called soft fillers. The construction is not difficult, and the preparation does not differ materially from that necessary for other pavements. Complete information as to the manner and method of executing the work may be obtained from the Bureau of Information of the National Paving Brick Manufacturers' Association, and is as readily obtainable as from any large commercial interest in the country on subjects vital to their commercialism. Mr. Blair.

Comforts in use with the cement-filled pavement necessarily follow the possible conditions resulting from such construction. An automobile ride on a cement-filled brick road is a luxury. As there are no waves and depressions, the short jolting effect is absent. The extraordinary durability insures an ever ready condition.

The speaker's advocacy of the grout filler is simply and entirely from the standpoint that it makes the best brick pavement; he cares not whether it be said that the grout protects the brick or the brick protects the grout, the fact remains that the protection afforded is sufficient for the pavement to continue to remain in a monolithic condition and smooth of surface—the desired and ideal condition.

The great merit of cement fillers is so far beyond that of other substitutes that, if expressed in words, would perhaps provoke a charge of extravagance. Let it suffice, however, to remark that, rather than hazard one's professional reputation, a careful examination into the proof of what is here said should be made by any one interested in this particular branch of engineering. Many hundreds of examples, ranging in age from 1 to 20 years, are afforded throughout the country, so that neither imagination nor theory need be utilized in reaching a conclusion.

H. B. Pullar, Assoc. Am. Soc. C. E.—It was not the speaker's intention to enter into this discussion of fillers for brick pavements, but as Kalamazoo and other cities in Michigan have been referred to, and as the speaker is familiar with the local conditions, he thinks it is only just to the engineers of these cities to explain their desire for a soft filler. Mr. Pullar

In Kalamazoo, a soft filler has been used almost exclusively because the city engineer, the city officials, and the property owners demanded it. They found an asphalt filler for use in brick pavements the most desirable because it made such pavements practically noiseless, sanitary, and easy to repair, three of the essentials for a brick pavement.

Mr. Blair has praised the cement grout filled pavements of Grand Rapids. Grand Rapids is within 100 miles of Kalamazoo, and is of approximately the same size. The pavements are subjected to practically the same amount of traffic, and are under the same climatic conditions, but the speaker does not believe that the city engineer of Kalamazoo, the city officials, or property owners are willing to admit that the Grand Rapids pavements are superior to those of their own city. Besides the places in Michigan referred to, there are more than

Mr. Pullar. 250 other cities in the Middle West which have used an asphalt filler with entire satisfaction, and these pavements are much superior, or at least equal, to the best of cement grouted pavements.

One of the greatest objections to a brick street has been the noise, and there is no doubt that, with a soft filler, this is abated to a very great extent. Each individual brick is thoroughly insulated, and the vibrations are confined to it instead of passing through the whole pavement, as is the case when such a pavement is grouted with cement. Such a pavement, on account of its monolithic nature, carries the vibrations for a great distance, causing a rumbling and making it even more noisy than one of concrete.

Mr. Blair states that one of the most serious objections to a soft filler is the fact that it does not protect the edges of the brick. Naturally, on account of the physical characteristics of soft filler, it is not capable of withstanding the action of traffic if the corners of the brick have already been chipped off. The filler is not supposed to act as a reinforcement for the brick, but only to fill the joints and act as a cementing medium. If the bricks are of the highest quality, a soft filler will thoroughly protect their edges, cement them together, and produce a water-proof and sanitary pavement, capable of withstanding traffic satisfactorily.

Another very important and favorable argument for the use of soft filler in brick pavements is the fact that immediately after the application of the filler, the street can be opened to traffic. When grout is used, it is necessary to close the street to traffic for from 7 to 10 days after it has been applied. This is very inconvenient, especially in making repairs, and for this reason the grout filler is looked upon with much disfavor by numerous municipal engineers.

The speaker does not doubt that there have been many failures of brick pavements in which soft fillers have been used, just as there have been numerous failures of such pavements with cement grout filler, but it is his opinion that at least 75% of these failures has been due to improper construction and poor application of the filler. In the use of a soft filler, as in that of a cement grout, it is very necessary that the utmost care be taken in the proper construction and in the proper application. The street must be clean, the interstices between the brick must be uniform and open, and the filler must be at a temperature hot enough to penetrate to the bottom of the bricks and thoroughly bind them together. Where the right kind of soft filler is used, and where it is properly applied, there is no doubt that first-class results can be obtained.

Mr. Blanchard. ARTHUR H. BLANCHARD, M. AM. SOC. C. E. (by letter).—In a discussion relative to the comparison of pavements from various standpoints, such as ease of traction, first cost, cost of maintenance, annual cost, etc., the properties of noiselessness and non-productiveness of dust are always considered. These are of the utmost importance under many different environments, and their attainment is expected by the public in connection with a general campaign for civic improvement. From these standpoints, is it possible to improve the brick pavement? It appears to the writer that the answer is in the affirmative. Mr. Blanchard.

By the use of the proper kind of bituminous material, a surface may be constructed which will not only render the brick pavement almost noiseless, but will also tend to decrease the quantity of free dust which may be raised from the surface by the passage of swiftly moving motor vehicles or strong currents of air.

If such a bituminous surface is constructed, it is obvious that the majority of municipal engineers would favor the use of a bituminous joint filler, because, with a filler of this kind, it is known that expansion and contraction and the rumbling noise characteristic of certain brick pavements having joints filled with cement grout are obviated. This opinion is based on the examination of many brick pavements throughout the United States, the joints of which, in some cases, have been filled with bituminous filler and, in others, with cement grout.

The bituminous surface to which reference has been made could be constructed economically by applying the asphalt cement to the brick surface with hand-drawn gravity distributors, using from ½ to ¾ gal. per sq. yd. This surface would be covered with a coat of stone chips, which would be rolled with a 5- to 8-ton tandem roller. It is apparent that the cost would be low, and, if distributed over the years during the period of its life as a factor of annual cost, would become negligible, when the benefits accruing from its use are considered.

Although surfaces of bituminous material on brick pavements are an innovation, bituminous wearing surfaces are not new, as sheet asphalt of varying thicknesses has been used on old brick pavements for a number of years in various municipalities. There appears to be no reason why the bituminous surface should not be as popular on the brick pavement as it is when used on the cement concrete pavement.

The writer has observed that some brick pavements which have been especially well constructed, and in connection with which a cement grout filler was used, are slippery, due to the glaze which is found on many first-class paving brick and to lack of foothold at the joints. Brick pavements having bituminous fillers are not as slippery as those with grout fillers. If the pavement has a bituminous surface, constructed with the proper kind of asphalt cement, it will be still less slippery, because the glazed surface of the bricks will not be exposed.

George W. Tillson, M. Am. Soc. C. E. (by letter).—The discussion on this question of joint-filling has shown plainly a difference in practice among municipal engineers in work of this class. It also Mr. Tillson.

Mr. Tillson. shows that one method has been successful in some localities and another method in others, which would prove very clearly the fact that, in the use of cement grout or bituminous filler, it is absolutely necessary that the work be done properly and with the proper materials. This is spoken of particularly in Mr. Kingsley's discussion, in connection with brick pavements, in the construction of which there seems to be the greatest variation in practice. If either a cement grout of proper quality, properly applied, or a bituminous filler of the proper characteristics, is used, good results will undoubtedly be obtained.

While the writer thinks very favorably of bituminous filler, he also thinks that it is more difficult to get proper results with it than with cement grout. Mr. Oxholm's discussion shows failures of two kinds of bituminous filler. If cement grout is applied so that the pavement does not crack, the writer believes that a much more durable pavement will be obtained than with the bituminous filler. If the surface of the pavement can be kept smooth, the actual wear of traffic is on the top of the bricks, so that the friction, and, consequently, the wear, will be less on the bricks themselves. There is nothing which increases the abnormal wear of pavements so much as a rough surface, no matter what the character of the pavement.

The instance noted by Mr. Whinery, in which a prominent member of the Engineer Corps of the Army stated that the best pavement for city streets is the one torn up the easiest and put back the easiest, does seem to be ironic. At the same time, there is some reason in the statement, as it does not make so much difference what the character of the original pavement is, if it must be torn up as frequently as some in American cities, if it cannot be properly and easily repaired. The true way, of course, is to prevent such openings as much as possible.

The method referred to by Mr. Sibley, as to the practice in St. Louis for laying wood blocks, is similar to that used in London and Paris, where the blocks rest directly on the concrete. It must be remembered, however, that the blocks used in Europe are softer and more resilient than the yellow pine blocks generally used in America. It means, too, that the concrete is finished with a much smoother surface than it usually receives in the United States; but that could be easily done if it were thought necessary.

Mr. Howell refers to the brick pavements in Kalamazoo, Mich., which he inspected some years ago, where both grout and bituminous filler were used, and, at that time, the sentiment seemed to favor the bituminous filler. The writer had an opportunity last fall to examine some of the brick pavements in that city and found them to be in first-class condition; and, while he undoubtedly did not see them all, the joints in all those which he inspected were filled with cement grout.

The writer thinks that perhaps the greatest benefit which has been derived from this discussion is the demonstration of the necessity of using the best material in the best way in filling joints in pavements, no matter what the filler or what the character of pavement, as it has been plainly shown that both cement and bituminous fillers have given first-class results. Mr. Tillson.

(3) BITUMINOUS SURFACES.

By Messrs. A. W. Dean, W. D. Uhler, Arthur H. Blanchard, William H. Connell, Fred. E. Ellis, P. P. Sharples, Clifford Richardson, Harold Parker, W. W. Crosby, J. A. Johnston, James Owen, C. J. Bennett, A. S. Brainard, G. Immediato, W. H. Fulweiler, Amos Schaeffer, and A. W. Dean.

Mr. Dean. A. W. Dean, M. Am. Soc. C. E.—In introducing this topic, in order that there may be no misunderstanding of the scope of the subject, it should be stated that the generally accepted definition of a bituminous surface is "a surface consisting of a superficial coat or coats of bituminous material, with or without the addition of stone, slag, gravel, sand, or other similar material," thus distinguishing bituminous surfaces from bituminous pavements, the latter consisting of bituminous material and stone, slag, gravel, sand, or other similar materials incorporated together.

Bituminous pavements of various types have been in use for many years, whereas bituminous surfaces are of recent adoption. The early superficial applications of oil were made largely for the purpose of dust prevention on dirt roads, a crude light oil being used, having very little binding quality. The advent of motor-vehicle traffic, however, has led to a very extensive use of bituminous surfaces, not only for the purpose of dust laying, but for the preservation of the roads. As a natural consequence, many experiments have been tried by road authorities to determine what methods and materials are best adapted to overcome the difficulties encountered, and by producers to determine what quality of bituminous material can be manufactured at a minimum cost to meet the requirements of the road authorities. Being still in the experimental stage, final conclusions regarding methods and materials are obviously impossible. It has been clearly demonstrated, however, that no uniform specification can be adopted defining a material which will produce a good bituminous surface on roads of every type and under every condition of traffic. Experience has shown, for instance, that while a heavy refined tar may be used to advantage on a macadam road, it is of no value as a surface application on an ordinary gravel or dirt road.

For surface treatment of dirt roads, a light oil helps somewhat to preserve the road, in that it prevents the particles composing the surface from blowing away, and assists, to some slight degree, in hardening the surface.

For surface treatment of gravel roads, the best results appear to be obtained by using an asphaltic oil of what might be termed medium viscosity, or by approximating the maximum viscosity that will permit application through an ordinary distributor at a temperature of 50° Fahr. Mr. Dean.

For surface treatment of broken stone roads, a light or medium oil acts mainly as a dust layer, yet if frequently applied it preserves the road to a very appreciable extent. In determining what bituminous material would be the most economical and advantageous for the preservation of broken stone roads by surface treatment, a knowledge of the traffic over the road is absolutely essential. If the road is subjected to light motor-vehicle traffic and light team traffic, with the motor vehicles predominating, experience has shown that an asphaltic oil, of such viscosity that it requires heating to at least 250° Fahr. before application, forms a bituminous surface which withstands the traffic and thoroughly preserves the road for a period of time depending partly on the quality of the material and workmanship and partly on the quantity of traffic.

Chemists do not agree unanimously on definite requirements for bituminous materials to be used for surface applications, and, as this method of treatment of roads is of such recent practice, it is probable that at least two years more must elapse before positive specifications can be drawn. Producers claim that the best oils for the purpose contain 90% asphalt. Although erroneous, it has become quite common to define asphalt oils in this manner, that is to say, by mentioning the alleged percentage of asphalt. When one considers the extreme heat which is applied to residuum oils in the process of manufacture, it is natural to form an immediate conclusion that the material may have become burned to a certain extent, and, consequently, be of quality inferior to oils derived by the use of natural asphalts. This may be true, but it must be proven in actual work before it can be accepted. Residuum oils placed on roads in Massachusetts early in the season of 1909, still show life and an indication of durability for a considerable time to come, and this fact would show that, while natural asphalts may possibly be superior, the residuum asphalts are, nevertheless, suitable for the purpose.

Fully as important as the quality of the bituminous material is the quality of the workmanship in applying it. In the preparation of the broken stone surface, extreme care should be taken to sweep and remove every particle of dust and dirt, so that the stones will be absolutely bare. Many failures of bituminous surfaces can be traced directly to the improper preparation of the broken stone surface, the heavy oils being distributed on dusty and dirty sections, and, consequently, peeling up through lack of adhesion. In order to get the best adhesion of asphaltic oils, it appears that the stone surface should also be somewhat moist rather than extremely dry. In distributing the oil, if the stone surface is comparatively new and smooth, the best results appear to be obtained by applying the oil under pressure

Mr. Dean. in two applications, each of ¼ gal. per sq. yd., covering the first application with grit or pea stone before putting on the second, and covering the second application with the same material as soon as possible after it has been made. The effect of applying the material in this manner is to make the distribution more uniform and prevent surplus oil from flowing on the sloping crown of the road, thereby causing ridges and bunches to appear after the work has been done. If the stone surface is full of slight depressions, however, a single application of ½ gal. per sq. yd., applied with or without pressure, has proved satisfactory. The oil tends to run to the depressions, causing a slight surplus of oil in them, so that when the grit is applied on top of the oil, the portions over the depressions absorb more grit, consequently rendering the road more smooth.

The character of the grit or other material used for covering the oil is of great importance. Where the traffic is confined exclusively to motor vehicles, sand appears to be as effective as any material for covering, but if there is some steel-tired horse-drawn traffic, a coarse material like pea stone or fine gravel is necessary.

The cost of a bituminous surface as just described will vary, of course, with the availability of the material to be used for covering, and the length of haul of all materials. In Massachusetts, during the last four years, several hundred miles of macadam roads have been improved or preserved by a bituminous surface of this kind. The average cost during 1910 was a little less than $0.08 per sq. yd., and, during 1911, a little more than that price, with labor costing from $1.75 to $2.00 per 8-hour day, and asphaltic oil costing $0.06 per gal. delivered in tank cars. The detailed cost on an average road is as follows:

	Per square yard.
Cleaning and sweeping	$0.0056
Patching old surface	0.0016
Cost of oil	0.0319
Heating oil	0.0031
Delivering oil	0.0038
Distributing oil	0.0029
Furnishing sand beside road	0.0165
Spreading sand	0.0073
Watering	0.0012
Rolling	0.0002
Supervision	0.0025
Total	$0.0766

The road mentioned was treated with ½ gal. of heavy asphaltic oil in two ¼-gal. applications. The average haul was 2 miles for the oil and 2½ miles for the sand. No allowance is made in the foregoing detailed statement for rental or depreciation of machinery, or for profits to contractor, the work being done by labor force account. Mr. Dean.

In maintaining these bituminous surfaces a re-treatment of about ¼ gal. of bituminous material per sq. yd. is only made on those places from which the bituminous material has disappeared. To show the probabilities of the cost of maintenance of roads by applying bituminous surfaces thereon, the speaker might cite 18 miles of State highway constructed in 9 towns in Massachusetts in 1909, the bituminous surface consisting of ½ gal. of residuum asphaltic oil. The first cost of the bituminous surfaces on these roads, in 1909, averaged $0.0742 per sq. yd. In 1910, there was expended for patching and sanding, $0.0146, and, in 1911, $0.0088 per sq. yd. The present condition of these roads indicates that the expense for patching and sanding in 1912 will not exceed $0.01 per sq. yd., in which case the total expense of maintenance of the surfaces on these roads for four years will have been $0.1076, making the cost $0.0269 per sq. yd., or approximately $236.72 per mile per year for a 15-ft. road, which cost does not exceed that of maintaining similarly located macadam roads previous to the advent of motor vehicles. Whether or not a bituminous surface, such as that just described, on a macadam road, will withstand the traffic of heavily loaded motor trucks cannot now be determined, as motor trucks have not been in use on such surfaces for a sufficient length of time and in sufficient numbers to permit such determination.

On roads where the prevailing traffic consists of steel-tired horse-drawn vehicles, this application of bituminous surface, consisting of heavy asphaltic oil and grit, has proved unsuccessful, in most instances, the surface being cut and dented to such a degree that it soon disappears. On such a road, it is possible that a heavy, refined-tar surface may be economical, or it may be economical to use oil of a lighter grade, applying it with sufficient frequency to keep the surface of the stone covered with oil at all times. The results, in the surface treatment of such roads in Massachusetts, would indicate that bituminous surfaces are not economical where the prevailing traffic consists of horse-drawn vehicles, but that a more durable construction of the crust of the road must be made by either mixing or penetrating the upper course of stone with bituminous material.

W. D. Uhler, M. Am. Soc. C. E.—Under the speaker's direction during 1911, more than 100 miles of water-bound macadam and gravel in the State Roads of Maryland were treated with a dozen varieties of asphalt oils and tars. Detailed information as to materials and quantities used, cost, etc., is submitted in Table 1. Mr. Uhler

As will be noted, the cost of these surface applications varied from 1.8 cents to 8.93 cents per sq. yd., or from $148 to $734 per mile; the

Mr. Uhler.

TABLE 1.—COSTS OF ROAD SURFACES TREATED WITH BITU- COMMISSION, MARYLAND,

County.	Name of road.	Con. No.	Length, in miles.	Width, in feet.
Allegany	Eckhart Mines-Garrett Co. L.	0140	2.86	14
	Std. Oil Warehouse Section	0142	1.25	12, 14 and 16
Caroline	Denton-Federalsburg	051	0.68	14
	Denton-Federalsburg	053	3.63	14
	Denton-Federalsburg	053	0.40	14
	Denton-Federalsburg	054	2.68	14
	Greensboro-Denton	055	4.07	14
Carroll	Sykesville-Eldersburg	0200	2.59	12 to 14
	Nicodemus	0206	1.00	12 to 14
Cecil	Conowingo-Porter's Bridge	040	1.06	14
	Conowingo-Porter's Bridge	040	2.06	14
	Rising Sun-Calvert	041	1.04	14
	Rising Sun-Calvert	041	2.22	14
	Elkton-Singerly	043	2.35	14
Charles	La Plata-White Plains	0150	4.63	14
Dorchester	Shiloh Church-East New Mkt	071	2.61	14
	Shiloh Church East New Mkt	071	0.138	14
	E. New Market-Mt. Holly	072	1.80	14
	E. New Market-Mt. Holly	072	4.10	14
Frederick	Jefferson Pike	0245	3.00	12
Garrett	Oakland-Thayerville	0161	5.61	14
Harford	St. Ignatius Ch.-Graf. Shops	0171	1.12	12 to 14
	Belair-Kalmia	0175	0.51	14
Howard	Baltimore-Washington, Sec. 3	01	1.18	12
Kent	Chestertown-Kennedyville	0120	3.28	14
Montgomery	Rockville-Gaithersburg	0230	2.01	12 to 14
Pr. George	Forestville-Marlboro	0130	2.07	14
	Dist. Col.-Charles Co. Line	0131	2.74	14
	Dist. Col.-Charles Co. Line	0131	1.28	14
	Dist. Col.-Charles Co. Line	0131	1.79	14
	Marlboro Road, Section 1	0138	1.31	12
Queen Anne	Centreville-Church Hill	0101	3.05	14
	Centreville-Church Hill	0102	3.14	14
St. Mary's	Mechanicsville-Leonardtown	020	5.34	14
	Mechanicsville-Leonardtown	021	3.49	14
Talbot	Easton-Wye Mills	0111	4.94	14
	Salisbury-Mardella Springs	080	1.80	14
	Salisbury-Mardella Springs	080	1.80	14
	Salisbury-Mardella Springs	080	1.29	14
	Salisbury-Mardella Springs	080	2.37	14
Worcester	Berlin-Snow Hill	060	4.56	12
	Berlin-Snow Hill	060	3.41	12
			102.258	

(1) Passing ¾-in. and retained on No. 8 screen.
(2) Sand, medium size.
(3) Washed gravel; average ⅜ in.
(4) Granolithic passing ⅜-in. and retained on No. 10 screen.

cost of bituminous material varied from 3.75 cents to 9.1 cents per gal., f. o. b. at the point of delivery; grit for the top dressing cost from 0.33 cent to 3.5 cents per sq. yd. in place, depending on its character and location. With the exception of 16 miles on which the bituminous material was applied under pressure, it was all applied with gravity oilers. While varying conditions will affect the figures slightly, a fair average of the detailed costs is as follows:

MINOUS MATERIAL, MAINTENANCE DIVISION, STATE ROADS JAN. 1ST TO DEC. 31ST, 1911. Mr. Uhler.

Trade name of bituminous material used.	Date of treatment.	Total cost.	Total cost per square yard.	Gallons per square yard.	Pounds of grit per square yard.	Character of top dressing used.
Fairfield No. 2	8/26– 9/20	$1 177.42	$0.0500	0.481	4.66	Limestone and sandstone chips (1).
Trinidad "A"	9/22–10/11	734.18	0.0694	0.660	17.06	
Asphaltoilene	6/ 2– 6/ 9	282.59	0.0504	0.500	......	No top dressing.
Ugite	5/22– 5/26	1 027.67	0.0344	0.427	7.84	Local sand (2).
Asphaltoilene	6/10– 6/13	171.31	0.0520	0.50	......	No top dressing.
Trinidad "A"	8/16– 9/ 4	899.74	0.0409	0.526	9.63	Local sand (2).
Ugite	6/ 5– 6/28	1 082.03	0.0324	0.410	6.73	Local sand (2).
Ugite	7/17– 8/12	1 023.49	0.0507	0.548	7.33	Pea gravel (3)
Trinidad "A"	7/28– 8/ 4	407.59	0.0525	0.570	7.50	Local sand (2).
Standard No. 5	6/ 7– 6/19	412.37	0.0473	0.526	11.32	Stone chips (1).
Tarvia "B"	6/ 7– 6/26	668.73	0.0395	0.474	......	No top dressing.
Standard	5/23– 5/30	305.29	0.0358	0.526	10.02	Chips (4).
Ugite	6/ 1– 6/ 6	788.38	0.0431	0.481	10.08	Chips (4).
Texas 60%	6/24– 7/ 3	880.86	0.0455	0.509	5.87	Local sand (5).
Ugite	7/26– 8/ 9	1 650.29	0.0434	0.600	9.07	Local gravel (6).
Texas 60%	7/20– 9/ 4	1 010.92	0.0471	0.533	6.00	Local sand (2).
Trinidad "A"	9/2	48.67	0.0427	0.438	6.00	Local sand (2).
Texas 60%	7/ 2– 8/10	646.63	0.0437	0.520	6.83	Local sand (2).
Trinidad "A"	7/22– 9/10	1 453.08	0.0431	0.553	5.80	Local sand (2).
Gulf-Asphalt "C"	8/ 1– 8/10	380.73	0.0180	0.380	......	No top dressing.
Asphaltoilene	6/24– 8/ 1	2 720.35	0.0591	0.500	......	No top dressing.
Texaco Special	9/29–10/31	550.40	0.0687	0.520	7.94	⅜-in. gravel (3).
Texaco Special	9/29–10/31	336.83	0.0801	0.498	8.33	⅜-in. gravel (3).
Standard No. 5	6/27– 6/28	240.70	0.0247	0.400	......	No top dressing.
Texas 60%	8/30– 9/21	1 076.80	0.0399	0.480	6.96	Local sand (5).
Trinidad "B"	10/12–11/ 7	1 367.78	0.0893	0.498	36.03	⅜-in. gravel (3).
Ugite	6/ 5– 6/21	747.64	0.0440	0.510	9.97	Local gravel (6).
Trinidad "A"	9/22–12/ 8	1 414.78	0.0627	0.490	15.20	Local gravel (6).
Ind. Refg. Co. Liq. Asph.	9/22–12/ 8	995.70	0.0948	0.707	30.43	Local gravel (6).
Fairfield No. 2	9/22–12/ 8	891.41	0.0606	0.442	12.40	Local gravel (6).
Trinidad "A"	8/14– 8/25	482.18	0.0522	0.544	11.50	Local gravel (6).
Ugite	8/14– 9/ 8	1 185.74	0.0472	0.518	9.93	Chips (4).
Ugite	9/27–10/14	1 092.40	0.0423	0.540	3.49	Chips (4).
Standard No. 5	6/30– 7/14	1 010.08	0.0270	0.333	3.68	Local sand (7).
Standard No. 5	7/ 3– 7/20	667.13	0.0272	0.333	4.15	Local sand (7).
Fairfield No. 2	9/ 5– 9/26	1 498.13	0.0369	0.515	......	No top dressing.
Trinidad "A"	6/13– 7/ 5	692.11	0.0468	0.54	9.00	Local sand (7).
Standard No. 5	6/13– 7/20	489.67	0.0331	0.54	9.00	Local sand (7).
Texas 60%	6/13– 7/20	652.33	0.0617	0.726	9.00	Local sand (7).
Fairfield No. 2	6/13– 7/20	655.23	0.0337	0.41	9.00	Local sand (7).
Asphaltoilene	10/ 7–10/28	1 737.75	0.0541	0.492	......	Sand from sides (7)
Ugite	10/ 9–10/28	969.20	0.0403	0.500	......	No top dressing.
		$36 526.30				

(5) Coarse sand.
(6) Unscreened gravel.
(7) Fine sand.

	Per square yard.
Sweeping	$0.0015
Pitch (delivered)	0.0300
Applying pitch	0.0045
Grit (delivered)	0.0030
Applying grit	0.0010
Total	$0.04

Mr. Uhler. In view of this experience, the speaker has arrived at the following conclusions:

(1) The road to be treated must be thoroughly swept before applying the bituminous material; otherwise the results will not be satisfactory.

(2) On a newly finished macadam road, about ½ gal. per sq. yd. will be necessary.

(3) In applying ½ gal. per sq. yd., it should be in two treatments of ¼ gal. each, wherever practicable.

(4) After applying the bituminous material, sufficient time (from 12 to 24 hours) should be allowed, when possible, for it to penetrate, and the road should then be covered with a light application of coarse sand, pea gravel, or granolithic chips. About 40 tons to the mile (14 ft. wide) have been found to be sufficient in most cases.

(5) In view of the experience with gravity and pressure distributors as to results, time, etc., it is thought advisable, as well as economical, to use a motor truck, fitted so as to apply the bituminous material under pressure.

Mr. Blanchard. ARTHUR H. BLANCHARD, M. AM. SOC. C. E.—During 1911 American manufacturers met the demand for low-priced distributing machines by placing on the market more than fifteen kinds, including both the pressure and gravity types. All these machines are more or less suitable for distributing one or more of the various kinds of materials used for the construction and maintenance of bituminous surfaces. They have not been calibrated for all materials, however, so that if the physical properties of the material, the temperature at which it is to be used, and the speed at which the machine travels, are known, the quantity they will apply per square yard can be definitely predicted. The speaker has found it advisable, therefore, before purchasing a machine, to secure reliable data covering the limitations relative to the character and quantity of material which it is capable of distributing, and the method and cost of operation.

There are, however, some well-known general limitations; for instance, it has been found extremely difficult with a gravity machine to distribute uniformly less than 0.4 gal. per sq. yd. with certain grades of material, unless the material is brushed into the road with brushes attached to the machine or in the hands of workmen. With some of the pressure machines, however, it is possible to obtain a uniform application with certain kinds of materials in quantities as small as 0.1 gal. per sq. yd. On the other hand, it has been found uneconomical to apply, with any of the 1911 pressure distributors, certain grades of asphalt suitable for the construction of bituminous macadam pavements or for the application of some asphalts, solid at air temperature, used for seal coats on bituminous concrete pavements.

Mr. Blanchard.

The hand-drawn gravity distributor will probably prove more economical and efficacious for the application of seal coats on bituminous concrete pavements than any other type of distributor. Since the seal coat is applied to the wearing surface as soon as a stretch of the latter is ready to receive it, the amount of work to be done in any one day is small, and would not usually warrant the use of a distributor of large capacity.

During the season of 1911 the speaker has noted a growing objection to the use of materials, for the construction of bituminous surfaces, which require from 2 to 6 weeks to set up to such an extent that tracking will not occur. By "set up" is meant that condition of the surface under which there is practically no tracking of the bituminous material or surface coat. During 1911 the speaker used several materials which have given satisfactory results from this standpoint. These include certain refined coal-tars and water-gas tars, combinations of asphaltic materials and refined tars, and an asphalt made of Gilsonite and asphaltic oil. It is reported that in Maryland certain cut-back Texas asphalts have given similar results. The following is an analysis of the Gilsonite asphalt compound, made in accordance with the methods proposed by this Society's "Special Committee on Bituminous Materials for Road Construction":

Specific gravity	0.98
Melting point of normal material, in degrees, centigrade	58
Solubility in carbon disulphide, percentage	99.7
Ash, percentage	0.1
Solubility in 88° Baumé paraffin naphtha, percentage	75.5
Fixed carbon, percentage	7.8
Viscosity, N. Y. T. L. viscosimeter, in inches	51
Penetration of normal material:	
No. 2 needle, 100g., 25° cent., 5 sec.	136
No. 2 needle, 200g., 0° cent., 1 min.	55
Evaporation, 5 hours at 170° cent., percentage	0.5
Melting point residue, in degrees, centigrade	62
Penetration of residue:	
No. 2 needle, 100g., 4° cent., 5 sec.	30
No. 2 needle, 100g., 25° cent., 5 sec.	102
Evaporation, 5 hours at 205° cent., percentage	3.4
Melting point residue, in degrees, centigrade	75
Penetration of residue:	
No. 2 needle, 100g., 4° cent., 5 sec.	19
No. 2 needle, 100g., 25° cent., 5 sec.	75

Mr. Blanchard. The speaker has found that, within 24 or 48 hours, bituminous surfaces constructed with the foregoing materials, using ½ gal. per sq. yd. and a thin covering of sand or chips, have set up so that no tracking is noticeable. Tar and tar asphalt compounds have long been recognized as having this property, but asphalts and asphaltic oils suitable for bituminous surfaces, from the above standpoint, have been difficult to procure.

Mr. Connell. WILLIAM H. CONNELL, ASSOC. M. AM. SOC. C. E.—The experience of 1910 in the Borough of the Bronx having proved that bituminous surface applications were more efficacious and economical than water sprinkling on macadam and earth roads, the water sprinkling division has been abolished, and all the macadam and a number of earth roads have received a surface treatment of tar or asphalt road oil. The results from tar have been very satisfactory, about ⅓ or ½ gal. per sq. yd. being applied and covered with torpedo sand or fine wash gravel. This formed a very desirable surface, at a cost of \$0.035 for ⅓ gal. and \$0.045 for ½ gal. per sq. yd. In these treatments the tar was applied cold.

The Grand Boulevard and Concourse was treated with a heavier tar, which was applied under pressure through a hose at a temperature of 220° Fahr., ⅝ gal. per sq. yd. being used, and then covered with torpedo sand or fine wash gravel. This road has been in use for 6 months, and although it has been subjected to very heavy, high-speed, automobile traffic, it is now in first-class condition. The cost was \$0.138 per sq. yd., which is high, owing to the lack of proper facilities for handling the bituminous material and the numerous delays which occurred. In the Borough of the Bronx a fair cost would be from \$0.09 to \$0.10 per sq. yd. for this treatment. Before the application of tar in these treatments, the road was thoroughly swept with horse-drawn and hand brooms.

Asphalt road oil of about 20° Baumé gravity was applied to a number of macadam roads, using ¼ gal. per sq. yd. On roadways having light or medium traffic, one application a year was sufficient to keep the road dustless; heavily traveled roadways required two and, in some instances, three applications. The cost of this treatment was \$0.013 per sq. yd. when ¼ gal. per sq. yd. was used. The oil was applied with a pressure distributor on a number of roadways, and the cost was \$0.009 for ⅓ gal. per sq. yd. This method of treatment is both economical and desirable. Just enough pressure was applied (about 15 lb.) to drive the oil into the interstices of the stone to a sufficient depth to avoid having a mushy road surface. Before the application of the asphalt road oil, the surface was swept with a horse-drawn sweeper only.

Preparations are now under way to equip the Bituminous Application Division with a sufficient number of pressure distributors to do all the bituminous surface work in 1912. For the cold treatments, the distributing device can be attached to an ordinary water sprinkler. The heavier materials will require the use of heater wagons. Considerable stress has been laid on this method of application, because, in the speaker's opinion, it results in saving not only labor, but also bituminous material, as the latter is applied to the road in such a manner that there is little chance of it being washed away. Mr. Connell.

The bituminous material applied to the different roadways was selected from the standpoint of its adaptability for each particular case. The stone used consists largely of Rockland Lake and Clinton Point crushed trap rock.

The following table gives a comparison of the cost of surface treatments and water sprinkling in the Borough of the Bronx, the water sprinkling being based on sprinkling from three to four times a day for 180 days, at $5 per day for a team, and water at $0.10 per 100 cu. ft.

Tar, $\frac{1}{3}$ gal. per sq. yd.	$0.035
" $\frac{1}{2}$ " " " "	0.045
Asphalt road oil, about 20° Baumé gravity, $\frac{1}{4}$ gal. per sq. yd.	0.013
Two applications	0.026
Asphalt road oil, about 20° Baumé gravity, pressure distributor, $\frac{2}{10}$ gal. per sq. yd.	0.009
Two applications	0.018
Water sprinkling	0.051

In order to compare these costs with those of other localities, the following figures relating to materials and wages paid to laborers and foremen in the Borough of the Bronx are submitted:

Foremen	$4.00 per day.
Laborers	2.25 " "
Average price of tar at freight yard	0.061 per gal.
Average price of asphalt road oil at freight yard	0.04 " "
Torpedo sand, on the work, but not spread.	1.30 per cu. yd.

With the use of pressure distributors in 1912, the cost of applying the tar will be greatly reduced. The present method requires the services of three laborers, whereas a distributor will need only one man to operate it, and moreover, the time required to apply the tar will be reduced to a minimum.

Fred. E. Ellis, Esq.*—In order to obtain the best results from the application of a bituminous wearing surface to a water-bound macadam road, it is necessary that the road be prepared so that the bituminous Mr. Ellis.

*Manager, Essex Trap Rock and Construction Company.

Mr. Ellis. material will adhere to the stone composing the surface. The unsatisfactory results obtained in the surface treatment of highways are not due in most cases to the bituminous material used, but to the character of the road and the manner in which it is prepared to receive the treatment. The speaker believes that a mistake is made by taking it for granted that a water-bound macadam road, constructed in the usual manner with a thin top course of small stone, is a proper surface on which to apply a bituminous coating. It is impractical to sweep the surface so as to make it entirely free from dust without at the same time making depressions where the small stones have been displaced by the broom. This is true where soft stone is used for the top course, and more especially where it is not uniform in character, as is generally the case when field stone is used. A comparatively thin top course composed of small stone is also objectionable for another reason. Vehicles, traveling on a bituminous surface which is inclined to be sticky, have a tendency to lift out the small stone and, in some places, to tear up the top course for its full thickness. This causes the small holes, so frequently seen in roads treated with a bituminous surface, which make such uncomfortable riding. If the dust or binder, either loose or compact, is not removed from the road previous to the application of the bituminous surface, the latter will push around on the road under traffic, and if it is not picked up by the wheels, it will soon lose its life and leave the dry macadam unprotected. When the macadam surface is exposed, the top course of the road disintegrates very rapidly, and, before the proper authorities are aware of it, the road is worn down to the bottom course.

It has been the experience of the speaker, and of others who have tried it, that if the top course of a water-bound macadam road is constructed of stones which vary in size from $1\frac{1}{4}$ to $2\frac{1}{2}$ in., with a depth of 4 in. after rolling, this layer being thoroughly filled with stone dust and flushed, such a surface will withstand traffic for a long time without raveling or breaking up. This surface can be swept clean without disturbing in any way the stones composing it, because they are large and are firmly embedded. There is also very little danger of the stones being lifted out by the traffic, due to the wheels sticking to the bituminous material. This method of construction is used in France, where most expert road builders and road users concede that the roads are the best in the world. The first expense of resurfacing an old macadam road in this manner will be somewhat greater than by the method ordinarily used. The additional expense, however, would seem to be justified because a great many macadam roads at present are not of sufficient thickness to withstand the heavy automobile truck and tractor traffic which, in a few years, they will be called on to bear.

In designing and constructing a road to receive a surface treatment, just as much care should be used in the selection of the stone and in the rolling and flushing as in the case of a road not to be so treated. The idea that a good road can be built with any and all kinds of stone or gravel, if it is to be constructed with a bituminous surface, is a false one, because the object of such a treatment is not to support the load, but solely to keep the binder in place. The cover for bituminous materials should be composed of broken stone screenings or gravel which will pass through a screen with a mesh of about the size of the thickness of the bituminous carpet required. This is necessary so that the weight of the traffic will be transmitted to the macadam by the stone composing the cover rather than through the plastic bituminous material itself. If the cover is composed of too fine material, without a mixture of a sufficient quantity of coarser particles, the surface will become rutted, the carpet rolling out very thin where the wheels run and increasing in thickness on each side where the traffic is not heavy. The bituminous material should be applied uniformly and in such quantity as will not cause the material to flow toward the shoulders of the road. This can be done best by a machine which applies the material under pressure. Where the bituminous material is applied in such quantity that it flows toward the shoulders, the surface will be wavy, because that part of the roadway where the flowing occurs will take up more of the covering material than where the flowing does not take place, thus giving a thicker carpet in some places than in others. Mr. Ellis.

These remarks do not refer to the application of the purely dust-laying oils. If they are to be used, the surface should not be swept so as to expose the stone, as by so doing the oil will lubricate the stone and the road will ravel.

P. P. Sharples, Esq.*—The speaker does not think that sufficient attention has been given to the difference in treatment required by the variation in the traffic on roads, and to other conditions. One who has followed the application of the tars and road oils which have been used in the United States during the past 6 or 7 years will have noticed that a product showing excellent results at one place may be a complete failure at another. This fact has often been attributed to differences in the bituminous material. A close study of many failures and many successes has shown, however, that the trouble is not caused by differences in the bituminous material, but by dissimilarity in the traffic and in the condition of the surface treated. Mr. Sharples.

At the present time our knowledge enables us to determine, in many cases, the proper treatment for a road. In many other cases we are not yet prepared to state definitely the best thing to do. Several general principles, however, may be deduced from the large number of experiments in New England. The heavy tars and asphalt oils are

* Chief Chemist, Barrett Manufacturing Company, Boston, Mass.

Mr. Sharples. suitable, as surface applications, when the traffic is mainly automobiles, and give excellent results. Such, at least, has been the experience on the main State highways in New England. When, however, to the automobile traffic is added a considerable amount of steel-tire traffic, the conditions change; and if, as is the case in the centers of towns and in the suburban districts of cities, the steel-tire traffic predominates, heavy bituminous surfaces are often failures. During the first rainy period, the steel-tire traffic and the horses' calks will probably cut up the surface, let the water into it, emulsify the bitumen, and produce a very disagreeable bituminous mud. It has been the speaker's experience that, for such traffic conditions, the thinnest possible treatments will give the best results. This is true of both tars and oils. The application must be made under proper weather conditions, and the dust, resulting from the traffic and from the bringing on of detritus from outlying and untreated streets, must be carefully removed before the bituminous material is applied. Careful attention should also be given to cleaning the streets after the application. Under these conditions light tars and light oils will be successful under traffic which would ruin thick bituminous coatings.

No general rule can be laid down for the time which should elapse between successive treatments. In the case of bituminous roads, the treatment, no matter what it is, must be renewed or the road patched as soon as there are signs of wear. This is a fact which is not yet realized by many engineers in New England, but the treatment of any bituminous surface must be looked after very closely, in order to insure economical results over a period of, say, 5 or 10 years. In some districts the treatment may last for 1, 2, or 3 years, while on another street in the same town, although the surface may have been prepared equally well for the reception of the bituminous material, the treatment will not last more than 6 or 8 weeks. After the treatment has once been commenced, it is folly to stop further applications and let the road go. The only economy is to continue the treatment and get the cumulative benefit of the applications.

In choosing the bituminous material for a surface which will have to be renewed often, care must be taken to select a material which will allow the application of repeated layers. The speaker has seen a number of roads where the material gave good results on the first treatment, but where further treatments added from year to year have produced a rolling and easily moved surface, due to the formation of a thick, plastic blanket. Where a road is to receive successive applications, it would seem to be important to choose a material which will set up or dry out sufficiently to give good results even after a good many treatments.

Another point in regard to these surface treatments, which has not been brought out, is the grade and shape of the roadway before treatment. The bituminous treatments, in every case, make a more slippery surface than the original bare macadam roadway, and, in planning for a surface treatment, this must be kept in mind. For any road having a bituminous surface, it is very important to reduce the camber or side slope to a minimum. A horse, in slipping, does not mind a forward or backward slip very much, but, if he slips sideways he falls at once, and this should always be kept in mind in the bituminous surfacing of roadways. Mr. Sharples.

The extensive application of bituminous materials to the surfaces of concrete streets has only come up within the past few years, but exceedingly good results have been obtained by the application of tar products to such surfaces. The tar materials seem to be especially adapted for this purpose, as there is no chemical action between them and the alkali of the cement, and the entire material adheres to the concrete until it is thoroughly worn out. The speaker has seen streets treated in this way, which have lasted for 2 years, even with moderately heavy traffic. At the end of this time the bituminous surface was worn down to a feather edge, but it was perfectly feasible to renew this surface at a small cost, and keep the concrete in good condition indefinitely.

Clifford Richardson, M. Am. Soc. C. E.—It may be of interest to explain why an application of oil to the surface of a macadam road after it has been watered may act better than on a dry road. There is always a slight coating of dust adhering to the surface stone, which prevents adhesion. If, however, the surface is sprinkled before the application of the oil, it converts this dust into a paste. The dust is the detritus of the rock, and, like clay, it is more or less colloidal. The result is that the dust in this condition will emulsify with the oil when the latter is applied to the surface, and will mix with it so readily that the bitumen will come in contact with the rock, and, after the evaporation of the water, will adhere perfectly. Clay and water will mix with any kind of asphaltic oil, and with the greatest facility. A great deal of it has been used on roads in Germany for distributing oil as an emulsion. The clay and water are mixed with the oil, put into the watering cart, and sprayed on the road. Mr. Richardson.

Another subject, to which the speaker would like to call attention, is the slipperiness of roads which have been treated with bituminous material. Some 25 or 30 years ago, when sheet-asphalt pavements were first being laid, they were objected to because they were so slippery that horses could not stand on them; even within the last year, the speaker has heard from a city in the State of Washington, where the City Engineer had decided that he would have no more sheet-asphalt pavement because it was too slippery. In the early days of the industry, General Edward Fitzgerald Beale, a great breeder of

Mr. Richardson. horses in Washington, D. C., in discussing the subject with the speaker, said:

> "That is due to the fact that the driver moving over a smooth surface does not drive with the same care as he would if he were moving over a rough surface, and it is also due to the fact that the horse has not learned to travel on that type of surface."

The speaker believes the latter is the better reason. In a certain section of New York State, one of the first bituminous roads, built some years ago, had rather too high a crown, and at first great objection to the character of the road was expressed by all the farmers in the neighborhood, because of its slipperiness in cold weather. Recently, the speaker happened to be in that locality, and asked persons living there what they thought of the road now. They said they had no objection to it; it was a perfect road. This would seem to show that it is a question of experience on the part of the horse and the driver as to how slippery a road may be.

Mr. Parker. HAROLD PARKER, M. AM. SOC. C. E.—Six or seven years ago, a concrete road about a mile in length was constructed with a bituminous surface of tar. That surface wore fairly well until two years ago; at that time it was treated with Tarvia, and is now in as good condition as when renewed.

Mr. Crosby. W. W. CROSBY, M. AM. SOC. C. E.—The speaker is very much interested as to the effect of water in the application of asphaltic oils to roads. Some bituminous materials which he has used for surface treatments have caused a disagreeable mud in the wet season. This mud seems to be due to the formation of an emulsion of the oil and water by the aid of the fine material, such as the finely divided clay, from the shoulders or from cross roads, brought on the treated road.

Some years ago, the speaker used considerable light oil for surface treatment, with the result that, in almost every case, the disagreeable mud complained of occurred during wet weather. Slight differences in the quantity or character of the mud led him to believe that possibly its formation was effected, not only by the traffic, but also by the character of the soil adjacent to the road in question. Consequently, he has been experimenting for a year or two along these lines, believing that light oils may give good results under favorable conditions. For instance, in these experiments, he has used, in both sandy and clayey localities, and under varying amounts of traffic, oils which have proved unsatisfactory elsewhere from the resulting muddiness. From the results of these experiments, he hopes to be able, in the near future, to prescribe limits, both as to traffic and clayiness of the adjacent soil, within which these as yet unsatisfactory materials may be used successfully.

The speaker cannot forbear to call attention again to the importance of recognizing the problem of each road as an individual one requiring particular, as well as careful, consideration for its solution. Mr. Crosby.

The adhesion of a bituminous material to a stone or concrete surface may be increased by the use of a pressure distributor. The pressure machine seems to act like the cement gun when used on dirty steel, because the sand blown through the gun against the steel cleans off the dirt and allows a good adhesion of the cement. In the same way, the pressure distributor seems to obliterate the dust film between the stone or concrete and the pitch, which nullifies the adhesiveness of the latter; at least, where it has often been difficult to obtain adhesiveness under a gravity application, the results have been entirely satisfactory where the same materials have been applied under pressure.

Five or six years ago, it occurred to the speaker that a cut-back pitch might have desirable characteristics, and he finally succeeded in getting from the Texas Company a cut-back product made up from an asphaltic cement of fairly hard (between 50 and 100) penetration, cut back with a light naphtha. The material was homogeneous, and thin enough to be applied cold. It was put on a new macadam road in good condition and gave excellent results. Since then, considerable of this product has been used with satisfaction. Recently, a product which resembles this original cut-back very closely has been placed on the market. It is capable of cold application, and, while the speaker does not remember all the details of the analysis, he thinks the material lost about 30% at 105° cent. in 21 hours. Each of these materials has been applied in quantities of about ½ gal. per sq. yd., and allowed to penetrate as much as possible under traffic conditions before coating with chips. About 40 tons of screenings per mile were used for coating, and in no case has any complaint reached the speaker concerning the "tracking" of the material. The resulting surface retained its elasticity and life for a considerable period. The cost was reasonable, the so-called natural pitch being cheaper than the manufactured cut-back. The latter required not more than 8 or 10 hours to set after the application of chips, while the "natural" article required about 18 hours.

J. A. Johnston, M. Am. Soc. C. E.—In Massachusetts, for three seasons, excellent results have been obtained by spraying bituminous materials with a pressure of not less than 70 lb. per sq. in., and in light coatings of ¼ gal. per sq. yd. With the nozzle used for this purpose, the pressure is not lost, for the bitumen strikes the road with such force that dust, leaves, or scraps are blown ahead of the machine and out of the way of the spray. This high pressure dislodges the dust and fine particles, forcing the bitumen into every Mr. Johnston.

Mr. Johnston. crevice and cranny of the road, adhering to, and gripping, the rough surfaces of the stone, resulting in a thorough bond.

It is conceded that gravity applications are not satisfactory, and that pressure is essential. Pressure below 50 lb. will neither dislodge the dust sufficiently to permit of proper penetration and adhesion, nor spray the light coats (of $\frac{1}{4}$ gal. or less) as uniformly as can be done with the greater force, and it has been the speaker's practice to use not less than 70 lb.

In repairing old macadam roads which were originally surfaced with No. 2 stone (by which is meant stone from $\frac{3}{4}$ in. to $1\frac{1}{4}$ in. in longest dimension), the surface was first scarified, shaped up, and then worked over with a light hand-harrow, or a farmer's weeder (the latter works even better than the harrow). This served to bring the stone to the top and shake the dirt down under it, so that a clear stone surface was obtained. On some of the roads built of soft stone there were sections so badly pulverized that there were no stone fragments to come to the top. Fresh stone was added in such places (but in 5 miles not more than 100 tons of new stone were used). The surface was then lightly rolled (just enough to smooth it out), after which it was sprinkled, and, while still damp, asphaltic oil, with a specific gravity of more than 0.98, and heated to 200° Fahr., was applied in a layer of $\frac{1}{4}$ gal. per sq. yd., with a pressure of not less than 70 lb. per sq. in., and then covered with grit. Two such coatings of bitumen and grit were used, making a total of $\frac{1}{2}$ gal. of oil, and 0.03 cu. yd. of grit per sq. yd. of surface, and the whole was well watered and thoroughly rolled. With this method the oil carpet sticks, does not creep, and does not roll up. When the surface was finished, the outlines of the stone fragments could be seen, but they were all covered, and thoroughly well bonded together with the bitumen, and none of these roads has shown any disintegration.

The speaker believes that, in applying a bituminous surface treatment to a newly built macadam road, it is best to scarify the surface, as previously detailed, but, if stone of large size (from $1\frac{1}{2}$ to $2\frac{1}{2}$ in. in longest dimensions) is used on the surface, it may be possible to sweep the road so clean that, without scarifying, a good adhesion or bond can be secured.

Ordinarily, it is good engineering practice to build structures which are so substantial that the depreciation and the maintenance cost are reduced, but, if the construction cost is made too large, the interest charge will be so great that it will more than offset the saving in maintenance.

To illustrate this: some time ago, an acquaintance asserted that there would be a great saving if all State roads in Massachusetts were surfaced with stone block or brick, because such paving could be laid in a permanent manner on a concrete base for an average of $3 per sq. yd., and, with the average travel to which the State roads are subjected, it would last 200 years, therefore the annual cost of maintenance would be only 1½ cents per sq. yd., which is much less than that at present. Mr. Johnston.

Of course, the fallacy of this is easily seen: for, to the depreciation charge of 1½ cents must be added the interest on the first cost of $3 per sq. yd., which at 4% is 12 cents, making the total 13½ cents, even with the impossible assumption that there would be no other maintenance cost.

Now there seems to be no doubt that, on most of the State roads in Massachusetts, water-bound macadam, covered with an oil blanket, can be maintained by applying a light treatment of oil (¼ gal. or less per sq. yd.) every 2 years, and the cost of this, with such small repairs as may be needed each season, averages about 3 cents per sq. yd. per annum.

The construction cost of such a road surface, including the bituminous top, is about 60 cents per sq. yd. The interest on this investment is 2.4 cents, and this, added to the yearly cost of maintenance and depreciation, makes a total of 5.4 cents for bituminized macadam, as compared with 13½ cents, which is an absurdly low estimate for the so-called permanent paving.

This difference of 7 cents looks small when one thinks of a square yard, but in a mile of road 15 ft. wide there are 8 800 of these square yards; this means that the yearly cost of the paving would be not less than $1 188 per mile, as compared with $475 per mile for the bituminized macadam. Moreover, the lower cost surface is dustless, noiseless, and has many desirable features which make for the comfort of the traveler and are entirely lacking in the stone block or brick road.

If proper methods are used in maintaining and resurfacing the bituminous coating, there need be practically no inconvenience to the traffic, for it is perfectly feasible to bituminize more than a mile of such a road in a day, and, in doing this, one side of the road can be kept open for travel.

Stone block, brick, and many other forms of paving, have their legitimate uses, but, in specifying the material or method for surfacing a road, the interest item must not be disregarded.

James Owen, M. Am. Soc. C. E.—Sometimes things are discovered accidentally and sometimes by afterthought. There seems to be a great deal of discussion as to whether a road should be cleaned or whether the bituminous material should be applied to the natural surface. The following is a rather curious instance: Oil for the repair of a road had been ordered, but its delivery was delayed. In the mean- Mr. Owen.

Mr. Owen. time, the road began to break, and, in order to save it from further disintegration, it was covered, according to the usual practice, with a coating of loam. This covering had been worn down by traffic for a couple of days when the oil arrived, and, contrary to the speaker's orders, was applied. That piece of road is now the best in the whole system. It would appear, therefore, that engineers have not yet sufficient knowledge to be able to predict just what the result of any treatment will be.

The speaker thinks that the formation of the mush on roads treated with oil is due to the fact that the oil-coated dust is too heavy to blow away and too slippery to wash away. The result is that, when the rains come, a complete and very efficacious emulsion is formed, which is sometimes 2 in. thick. It is difficult to eliminate this condition, and, in the speaker's opinion, about the only way to prevent it is to apply the oil in time for it to disappear before wet weather comes.

Mr. Bennett. C. J. Bennett, Esq.*—In Hartford, Conn., nearly all surface treatments with heavy asphaltic oils have been failures. The speaker thinks this is due to two things: the preponderance of horse-drawn vehicles on the city streets and the large quantity of clay soil which, in the fall, mixes with the oil and makes an emulsion resulting in a very disagreeable mud. This condition has obtained with the use of any asphalt oil, whether light or heavy, and whether applied under pressure or by gravity. The question in Hartford is whether the benefits from the oil as a dust-layer in the summer will counterbalance the disagreeable features in the fall. A Texas, 65% oil was used on streets with a heavy traffic until the fall with good results. Standard oil, 40%, Texas oil, 35%, and Indian oil, both light and heavy, were also used with similar results. Tarvia B was used on one street for a surface treatment, and, though it gave very good results, as far as the preservation of the surface was concerned, it was not a successful dust-layer, and therefore a light oil was afterward applied to the surface.

Mr. Brainard. A. S. Brainard, Assoc. M. Am. Soc. C. E.—The maintenance of the State macadam roads in Connecticut is accomplished by first applying a coat of Glutrin. After this has been allowed to season somewhat, a light coat of asphaltic oil is applied over the surface, which is then covered with sand or light gravel to prevent tracking.

The speaker is not prepared to state with assurance just what satisfaction this method has given, but it is argued by the Highway Commissioner that, by applying the Glutrin to the surface, he protects the road metal from the disastrous effects of the high-speed automobile. This method allows the aggregate to cement of its own natural ability,

* Superintendent of Streets, Hartford, Conn.

Mr. Brainard.

and, at the same time, protects the road from the lubricating action of the oil when it is applied, which is said to cause the surface to ravel and disintegrate.

In wet weather the roads are apt to mush up, but whether this is due to the quality of the oil used or to the quantity applied, the speaker is not prepared to say. Very little tar or other bituminous material has been used in construction, and only to a very small extent in making repairs.

Mr. Immediato.

G. IMMEDIATO, Assoc. M. Am. Soc. C. E.—The speaker will describe a failure which resulted from using asphaltoiline, at the rate of about ½ gal. per sq. yd., on a street with different grades, but subjected to the same traffic on all sections. The street carries the heaviest traffic in Montclair. On the hillsides, where the water was shed from the surface quickly, this treatment gave very good results; but on the low portions, where the water could not get away as quickly, the surface mushed up. The asphaltoiline was applied hot (175° Fahr.), after the street had been thoroughly cleaned and scraped. As soon as the oil was put on, it was covered with screenings—about 100 lb. per sq. yd. The surface was thoroughly rolled, and the street was closed to traffic for two days. This gave the material a chance to set thoroughly, which it seemed to do on the hillside. On the low portions, however, it did not set, but remained soft until the end of the season. After every rain the mud was 6 or 8 in. deep in some places, and, about 6 weeks ago, it was necessary to scrape the street and remove this mud.

On one street the speaker used calcium chloride in the first part of April, and about 5 weeks afterward the surface was oiled with No. 4 Standard Oil, heated to 175° Fahr. This application was covered with a light coat of screenings, and rolled. This treatment was found to be sufficient to allay the dust throughout the whole season, or from 6 to 7 months. The only streets in Montclair, however, which are absolutely mudless were constructed with Tarvia. One street broke up in August, 1910, and was rebuilt in the following spring by the penetration method, a mixture of Tarvia A and Tarvia X being used.

Mr. Fulweiler.

W. H. FULWEILER, Assoc. M. Am. Soc. C. E.—When the steel-tire traffic becomes pronounced on a road or street, the thinnest possible surface treatment seems to give the best results. The speaker has found that an application of about ½ gal. of fairly heavy binder covered with screenings would give good results where automobile traffic was predominant; but when that same treatment was used in localities where the character of the traffic was similar to that in a small town, the surface was cut up badly, and the best results were obtained by a very thin treatment which simply protected the surface, Mr. Fulweiler. or acted like a coat of varnish on a floor. This treatment is applied directly on the stones which have to carry the load.

It seems highly probable that, if two applications are better than one, three are better than two. If it is attempted to apply a cold material with a gravity distributor, it is very difficult to put on much less than ½ gal. per sq. yd. If a pressure distributor is used, a very much smaller quantity can be applied. If a total of about ⅜ gal. per sq. yd. is put on in three separate applications, the result is apparently quite as desirable as with the single application of ½ gal. with the gravity sprinkler, and it is much more uniform. In this way, if a spot is missed on the first application, the second treatment covers it, or if it is missed on the second treatment, the third treatment covers it. By applying three coats and by allowing 8 or 10 hours for each coat to set, the best results are obtained, and ⅛ gal. per sq. yd. is saved. It does not cost any more to apply the material in this way; it simply means that the distributor is driven over a greater distance, and the actual time consumed in applying the material is not the important factor. The time is lost mainly in loading the distributor, hauling it to the road, and getting it ready for work.

With regard to making applications with pressure machines: If a heavy pressure is used, it will apparently atomize the bituminous material, and when this happens, it ceases to strike the road with that necessary, directional velocity which blows the dust away as the material is distributed. The nozzles of the machine with which the speaker is most familiar are similar to the flat-top Bray burners used for illuminating gas. As they leave this nozzle the two streams of material impinge on each other and form a flat sheet or spray of material at right angles to the original plane of the two streams. The amount of pressure used modifies the shape of the spray. As the pressure is increased, the sheet of material strikes the road with increasing force, and blows the dust from the surface very effectively. If the pressure is increased still further, a point will be reached that will cause the lower edges of the spray to open or separate, and at this point the material has become atomized. From this point a further increase in pressure will more completely atomize the material until finally it is all in that condition as it leaves the nozzle, and reaches the road in minute drops rather than a solid sheet, actually defeating the desired scrubbing action on the surface.

The speaker has found that the requisite pressure varies with the nature of the work and the class of material used. In penetration work, where the stone in the road is reasonably free from dust, enough pressure is put on to atomize the material. In one type of machine about 70 lb. is used to get a good atomization. The material is of heavy grade, is heated to about 250° Fahr., and is applied at the rate of about 1½ gal. per sq. yd.; but, when it is to be applied at the rate of

$\frac{1}{8}$ gal. per sq. yd. for surface treatments, the pressure is reduced to about 25 lb., and then the material, instead of leaving the nozzles as a mist, leaves as a solid sheet. If there is any breeze at all, it is particularly important to apply the material as a sheet and not as an atomization. Mr. Fulweiler.

When it is desired to atomize the material, a somewhat different type of nozzle is used, which delivers more of the binder than the ordinary form. It is difficult to apply $1\frac{1}{2}$ gal. per sq. yd. through a simple nozzle, unless the machine is run very slowly. The other nozzle works on the same principle, but, instead of having two jets impinging on each other, there are eight pairs of jets, thus giving eight times the delivery per nozzle.

Amos Schaeffer, M. Am. Soc. C. E.—Mr. Blanchard has classified the methods of constructing street surfaces as surface treatments, bituminous macadam pavements (those built by the penetration method), and bituminous concrete (those built by the mixing method). The consensus of opinion seems to be that the surface treatment of water-bound macadam is very successful. The same conclusion seems to have been reached in reference to bituminous concrete pavements. It has also been shown that there have been a great many failures of pavements built by the penetration method. The result of the speaker's observation of surface treatment of water-bound macadam in the Borough of the Bronx is that it is relatively more successful than the penetration method. Mr. Schaeffer.

During the summer of 1910, the surface treatment of water-bound macadam was started with the lighter oils, and a little later in the season the heavier oils were used. The light oils acted simply as dust layers. The later use of the heavy oils demonstrated beyond doubt that they were more economical as dust layers and also acted as a binder for the road metal near the surface. All the oiling during the first season was more or less experimental, and the Maintenance Bureau was not properly equipped to do the work. When the heavy oils were used, therefore, there were neither sand nor stone screenings available to cover the oil after it was spread on the road, with the result that there was much complaint because it was dragged on the sidewalks and into houses. The condition of the roads on which the heavy oils were used, however, was very promising. A thin crust had formed over the surface, which was partly impervious and protected it from the rapid disintegration which obtains on water-bound macadam under traffic. All the streets treated in this manner were in very good condition in the spring of 1911.

During the following season, a coat of the heavy oil was again applied, over which was spread a thin layer of pea gravel. This remedied the nuisance of the previous year of dragging the oil on

Mr. Schaeffer. the sidewalks and into houses, and also made a better surface. The macadam pavements treated by this method gave better results than bituminous macadam. The bituminous macadam pavements with which the surface-treated pavements are compared were laid under the supervision of the representatives of the manufacturers of the different kinds of bitumen. It is to be presumed, therefore, that proper precautions were taken to insure the best results. It must be admitted, on the other hand, that the apparatus and labor were rather crude, and possibly better results could be obtained with better equipment and more skillful labor.

The good results obtained by the surface treatment of water-bound macadam are probably due to its greater compactness and the smaller percentage of voids which are the results of continuous traffic. It must not be inferred from this, however, that a long period of traffic is necessary before the application of oil is made. A considerable portion of The Concourse, in the Borough of the Bronx, was resurfaced with water-bound macadam and then opened to traffic for a month or six weeks, after which oil was applied to the surface and covered with a thin layer of pea gravel in the same manner as any other water-bound macadam. This treatment has likewise given better satisfaction than the bituminous macadam, although the traffic was considerably heavier than that on the former.

Since this experience has been shared by engineers in other parts of the country, it seems to the speaker that consideration of abandoning the penetration method entirely might be in order.

Mr. Dean. A. W. Dean, M. Am. Soc. C. E. (by letter).—It is apparent that all who have directed and watched the use of bitumen as a superficial surface treatment are agreed that, under ordinary conditions, pressure is quite necessary for the proper distribution of the material, and that two applications, each of $\frac{1}{4}$ gal. per sq. yd., are quite sufficient for a bituminous surface on a broken-stone road. If "hot" asphaltic oil or tar is used, a muddy surface cannot be produced on a good broken-stone road unless an emulsifying material is present, either in the broken stone or in the material used to cover the bitumen; and, moreover, it is impossible to conceive of a depth of more than 1 in. of mud, as there can be only a maximum of approximately 1 in. of material absorbed by or mixed with the bitumen. A fact which should not escape attention is that an excess of bitumen is detrimental, will result in creeping and rutting, and will create a very uneven surface in warm weather.

One matter which has not received much notice in the discussion is the renewal of bituminous surfaces. Where a hot-oil application has become partly worn off, or has otherwise disappeared from the surface, the mistake should not be made of giving the entire road a new application of ½ gal. per sq. yd., covering the old bituminous surface which has remained intact as well as the sections which have disappeared. This method of retreatment tends to have the same effect as that caused by the application of an excess of bitumen in the first place, the bitumen in the old surface becoming fluxed, or, as sometimes described, having new life imparted to it, thereby causing an uneven surface on account of the excess of bitumen on portions of the road. It has been found that, in order to secure an even surface in renewing an old bituminous surface, it is best to apply bitumen first to the sections where the old surface has entirely disappeared, and then treat the entire surface with a very light application. Mr. Dean.

(4) USE OF BITUMINOUS MATERIAL IN PENETRATION AND MIXING METHODS.

By Messrs. Linn White, Clifford Richardson, A. F. Armstrong, E. H. Thomes, F. C. Pillsbury, Arthur H. Blanchard, W. W. Crosby, G. W. Tillson, R. B. Gage, Frederick Dunham, H. C. Poore, Theodor S. Oxholm, J. W. Howard, Michael Driscoll, Herbert Spencer, P. P. Sharples, H. L. Collier, William H. Connell, T. Hugh Boorman, and A. W. Dow.

Mr. White. Linn White, Esq.* (by letter).—In offering this contribution on the use of bituminous materials in road construction, it is believed that safe conclusions can be drawn in such matters only from experience and direct observation. Therefore, what is offered will relate to work in Chicago and vicinity, particularly to the road work in the South Park System of that city.

The South Park Commissioners have under their control about 60 miles of drives, including various boulevards and park drives, all lying within the southern division of the city. The traffic over these drives varies from a maximum of 17 000 vehicles per 24 hours, of which 4 000 are heavily-loaded traffic teams, which extreme condition occurs on the northern end of Michigan Avenue, down to a minimum of a few hundred vehicles on the more remote boulevards, which carry no through traffic. The comparatively few miles of boulevard in the down-town district carrying the heavy down-town traffic, such as the northern end of Michigan Avenue, alluded to above, and Jackson Street, have long been paved in a substantial way, and will not be referred to further. Of the 60 miles of drives in the system, 90% is paved either with plain water-bound macadam or a bituminous wearing surface supported on macadam. The water-bound macadam surfaces are being maintained in a usable condition with as little cost as practicable until such time as they can be given a bituminous wearing surface. The through traffic on the principal park drives and various boulevards through the residential district, at an average distance of 6 or 7 miles from the center of the city, amounts to from 3 000 to 5 000 vehicles per 24 hours, in average fair weather, but this may be greatly decreased during bad weather conditions or greatly increased on Sundays and holidays during fine weather. As traffic teams are excluded, at least 75% of the vehicles are automobiles, but, on the numerous intersecting streets, the general traffic moves unrestricted across the boulevards, and must be added to the foregoing figures and reckoned with in considering the effects of traffic on the

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* Chief Engineer, South Park Commission, Chicago, Ill.

pavement surfaces. These brief statements should serve to give a general understanding of the conditions under consideration. Mr. White.

In 1905 bituminous treatment for preserving the macadam surfaces was begun, in an experimental way at first, but, under the comparatively dense traffic conditions, experience was rapidly gained. The emulsions and light road oils which were tried were quickly abandoned because of constant complaint from residents about the resulting sloppy, disagreeable conditions of the drives, as well as the failure of these materials to add much, if any, bonding value to the surface. Most of the macadam was of limestone, of none too hard a quality, which wore and disintegrated rapidly under the increasing traffic, and required something of more positive cementing value than could be obtained from the lighter forms of bituminous material. The first apparent success was with Tarvia of the grade afterward sold as "A." In the first experiments the tar was applied to the completed and well-filled surfaces of the macadam and covered with a top dressing of limestone screenings, thus securing a well-defined cushion coat and such penetration as good luck might bring.

In 1906 and 1907, 290 000 sq. yd. in the South Park System were surfaced in this manner, at an average cost of 5.7 cents per sq. yd. for the bituminous surface alone, and 19.5 cents per sq. yd. including the necessary re-dressing of the macadam and the addition, in some cases, of variable quantities of new stone. The quantity of bituminous material used was from ⅓ to ½ gal. per sq. yd. The results were variable and not always encouraging. When the macadam was somewhat dirty, or had an excess of fine material in its composition, the penetration was indifferent, and the surface scaled off, sometimes within a few weeks. The first improvement was to use clean limestone chips for the top dressing, and the next was to finish up the macadam so that there would be plenty of surface voids to secure penetration. In 1907, the necessity and means of securing penetration were beginning to be understood, and the work of this year was much more successful than that of 1906.

The first section of work done in Chicago, following what are now recognized as correct penetration methods, was in August, 1907, on Michigan Avenue from 41st to 42d Streets. Here was used a specially prepared grade of Tarvia, of about 1.20 gravity at 60° Fahr., corresponding to what was soon afterward put on the market as Tarvia "X." The quantity used was 1¾ gal. per sq. yd., and the cost was approximately 31 cents per sq. yd., including 1½ in. of new stone. This produced a fairly substantial surface, but it was torn up in 1909 on account of carrying through a uniform improvement. It was not worn out after 2 years of service, but had developed some disintegrated spots, and had the fault of bleeding under the hot summer sun, leaving it sticky when hot and glassy when cold. If pro-

Mr. White. vision had been made for a somewhat deeper penetration, or if more fine material had been rolled in when first constructed, so that there would not have been as much bituminous material near the surface, this condition might have been improved.

Other materials than Tarvia were used for penetration work, principally oils with heavy asphaltic contents requiring about 200° of heat for application. One section of Grand Boulevard surfaced in this way with California maltha of 1 gal. per sq. yd., at a cost of 23 cents per sq. yd., without the use of any new stone except the top dressing of stone chips, waved and rutted so badly under traffic as to be at times almost impassable. Repeated rolling, with a liberal application of screenings, failed to cure the defect, and the road was torn up in 1909. An analysis of the trouble indicates that it was due to insufficient penetration in the old and dirty macadam, and to the presence of too much fine mineral matter. On a layer of clean coarse stone the results would undoubtedly have been more satisfactory.

In 1908, about 200 000 sq. yd. of penetration work were laid, 10 000 sq. yd. being laid by the mixing method, following some previous small experiments. Some of the penetration work of 1907 was given a second coat of Tarvia of about $\frac{1}{3}$ gal. per sq. yd., which had the effect of adding a cushion coat without securing any penetration except where the surface was disintegrated. A second and even a third coat may be added with success, if there is a substantial structure of sound stone beneath; otherwise a shifting, unreliable surface will be obtained. Of the penetration work done in 1907 and 1908, amounting to 350 000 sq. yd., there remains in use about 100 000 sq. yd., of which 34 000 sq. yd. will have to be replaced with a new surface in 1912. None of the 1905 and 1906 work remains, and none of the 1907 and 1908 penetration work on street intersections, where miscellaneous traffic crosses the boulevards. All the 100 000 sq. yd., mentioned as being still in service, is where traffic is restricted and is principally rubber-tired.

As previously stated, in 1908 the first surfacing was done by the mixing method, amounting to about 10 000 sq. yd. No satisfactory machinery seemed to be available for mixing, unless standard asphalt paving plants were considered, and hand-mixing was recognized as being too expensive and slow. At the beginning of the season of 1909, there was on the market a portable plant manufactured by the Link Belt Company, of Chicago, which embodied all the essential parts of an asphalt paving plant mounted on one truck. The South Park Commissioners purchased two of these plants, which have been kept in use during the seasons of 1909-10-11, and, up to date, about 435 000 sq. yd. of mixed bituminous wearing surface have been laid with them, principally on the old macadam. A comparatively small amount has been laid on a new macadam base, but the method of construction was the same. The methods of doing the work have progressed from experiment to experience, and from experience to what may be considered established practice, in a process of evolution somewhat similar to that previously described for the pouring and penetration methods. More attention is now given to preparing the base with a coarse, open, and grainy top, into which the surfacing mixture may be forced. In the earlier work the macadam surface was left comparatively smooth, and dependence was placed on the composition of the mixture and the stability of the binder to prevent the shifting of the surface. Mr. White.

The bituminous wearing course in most of the work has been made 2 in. thick, in some 1½ in., and in a small quantity about 1 in. The later conclusions are that a comparatively thin and completely water-proof wearing surface, with a strong, coarse layer of stone keying it to the macadam base and giving lateral stability, is the most logical, economical, and successful construction, the layer of coarse stone corresponding somewhat to the binder course in sheet-asphalt pavements. The thickness of the wearing surface may be varied in the judgment of the engineer to meet the stress of traffic and amount of money available.

Careful records of cost have been kept, and may be concisely stated as follows:

	Labor.	Materials and supplies.	Total.
2 in. thick	$0.16	$0.36	$0.52
1½ " "	0.14	0.28	0.42
1 " "	0.12	0.20	0.32

These figures do not include the preparation of the base, overhead charges, or interest and depreciation. The same is true of any other figures of cost mentioned by the writer. The cost of labor and supplies in Chicago probably does not vary materially from that in any other large city in the United States, and the costs given above are based on the following average prices of materials used in the wearing surface:

Road asphalt	$20.00	per ton.
Crushed limestone	1.65	" cu. yd.
Sand	1.50	" cu. yd.

The bituminous cements used have been asphaltic compounds, natural asphalts, and, to a limited extent, refined tar of from 1.20 to 1.29 gravity at 60° Fahr.

All the surfacing done by the mixing method during the past 4 years, beginning with 1908, is giving good service with less than one-half of 1% of repairs up to date. Some weaknesses of construction have developed at certain points where the stress of traffic has been the greatest, and have caused the small amount of repairs

Mr. White. referred to. On some of the earlier pieces of work, surface cracks or checks have appeared, but it is believed that the better methods of construction, particularly the use of the coarse, open-stone base, have to a large extent overcome this tendency. The two other Park Commissions in Chicago, the West Park Commission and the Lincoln Park Commission, have adopted the same methods for surfacing macadam, and are using the same machinery for mixing and preparing the material. Altogether, there has been laid in Chicago by the three Park Boards nearly 1 000 000 sq. yd. by the mixing method.

In its meetings of the last three years the organization of City Officials for Standardizing Paving Specifications has adopted specifications for work of this class, under the name of "Bituminous Concrete," based principally on Chicago practice.

In conclusion, it may be said that the additional cost of surfacing by the mixing method over first-class construction by the penetration method is so slight, and the advantages of uniform results and longer service attained are so considerable, that every municipality should look carefully into it before deciding on the inferior method. It is probably true that it should not supersede the penetration method in all cases, but where there is a growing traffic, urban, suburban, or interurban, it will be the better method of construction.

Mr. Richardson. CLIFFORD RICHARDSON, M. AM. SOC. C. E.—It has been the speaker's experience that for the construction of bituminous macadam roads a hard limestone will prove extremely satisfactory. He has in mind two limestone roads constructed in New York which have shown certain characteristics, but which are preferable to those built of trap rock. This may be accounted for by the fact that a trap-rock fracture is a very glassy one. If any bituminous material should be poured on a sheet of glass, such as ordinary window glass, it could be torn away from that surface much more readily than if the surface was of ground glass. A limestone fracture, being somewhat granular in appearance, resembles ground glass more than trap rock, so that a better adhesion of the bituminous material results with limestone than with trap rock. Moreover, limestone under pressure produces a certain amount of detritus or dust, which acts as a better filler than the dust of trap rock. Good hard limestone has a better fracture than trap rock, especially Hudson River trap, which is another reason why the former is more desirable. As long ago as 1837, when Gillespie published his book on highways, he called attention to the fact that Hudson River trap rock is not as desirable as that in many other parts of the country.

The speaker has looked into the matter of stone-crusher products very carefully. Stone quarries are equipped with screens which cannot be changed to suit the convenience of engineers. The sizes of screens which have been suggested for future specifications are not just what will be found at the plants of the various quarries.

In constructing roads of gravel and sand the speaker has found it desirable, in a number of cases, to ascertain the voids in the gravel in order to know the exact quantity of sand necessary to fill them. In order to obtain a satisfactory compaction, about 5 or 10% more sand must be added than the voids in the gravel demand. Mr. Richardson.

The great difficulty with the work of this type which has been done is that the pavements have a tendency to bleed, but if the voids in one lot of gravel are carefully calculated, and if the variations in the bank are not too great, the results will be successful. One of the experimental sections of the White Plains Road in the Borough of the Bronx was constructed in this manner, and shows the possibility of doing successful work.

It has been stated that bituminous work should not be carried on after September 30th, or when the air temperature is below 50° Fahr. This is a very wise precaution, although the speaker thinks that the temperature might be reduced to 40° and the fixed date omitted. For instance, during 1911, work could be satisfactorily done in October, whereas, in the year before, the weather conditions were such that no satisfactory work could be done in that month. Many of the defects of bituminous roads have been due to the fact that work is done in the fall, and an opportunity of at least a month during warm weather should be given to the contractor to finish up his work for the season, no matter whether the road is being built by penetration or mixing methods.

A. F. Armstrong, M. Am. Soc. C. E. (by letter).—The New York State Highway Department built about 1 300 miles of bituminous macadam highways by the penetration method during 1909, 1910, and 1911. General observation of these roads, which have now been open to traffic for periods varying from 1 to nearly 3 years, indicate that they are in good condition, are giving excellent results, and apparently will continue to give satisfaction for some time to come. Mr. Armstrong.

There have been some failures, but they have been few. The writer estimates them as less than 1% of the mileage of the highways built. They were due principally to pouring too late in the season, poor workmanship, wet or dirty stone, brittleness in the bituminous material, and poor foundation. A trap rock road is naturally the hardest to bind, and raveling occurs more frequently on those built with this material than on those where other kinds of stone have been used. It is believed to be of the greatest importance that bituminous macadam, made by the penetration method, should be laid early enough in the season to have traffic over the road for at least 1 month of warm weather, so that it may be thoroughly compacted before cold weather arrives to harden the bituminous material.

During the season of 1910 cost data reports were received from a large number of roads. These were examined, and computations and

Mr. Armstrong.

TABLE 1.—(*Continued.*)

Highway No.	Length treated, in miles.	Width treated, in feet.	Bituminous Material.		Square yards spread.	Average Number of Gallons per Square Yard.		Average Cost per Square Yard.				
			Cost per gallon at road.	Gallons poured.		First pouring.	Second pouring.	Labor.	Materials.	Equipment.	Engineering.	Total.
Bermudez Asphalt Fluxed.—Application Made in Two Pourings.												
571........	2.32	14	$0.126	38 190	19 016	1.35	0.65	$0.047	$0.277	....	$0.014	$0.334
665........	1.01	12	0.125	12 487	7 094	1.27	0.49	0.061	0.263	....	0.010	0.334
686........	0.57	14	0.127	8 158	4 652	1.17	0.58	0.072	0.276	....	0.023	0.371
762........	4.70	16	0.118	77 507	44 242	1.25	0.50	0.054	0.230	....	0.010	0.294
867........	0.97	16	0.120	15 542	8 890	1.22	0.53	0.023	0.232	....	0.005	0.260
Totals....	9.57	......	......	151 884	83 894	....	....	....	....	....	....	......
Averages.	1.91	14.4	$0.123	....	....	1.25	0.55	$0.051	$0.256	....	$0.012	$0.319

Mr. Armstrong.

TABLE 1.—Cost of Bituminous Material and Filler used Therewith, in Place Complete, by the Penetration Method, Season of 1910. Department of Highways, State of New York.

Highway No.	Length treated, in miles.	Width treated, in feet.	Bituminous Material.		Square yards spread.	Average Number of Gallons per Square Yard.		Average Cost per Square Yard.				
			Cost per gallon at road.	Gallons poured.		First pouring.	Second pouring.	Labor.	Materials.	Equipment.	Engineering.	Total.
Residuum Products.—Application Made in One Pouring.												
321........	4.35	16	$0.110	61 278	40 890	1.50	....	$0.035	$0.180	....	$0.004	$0.219
393........	3.58	16	0.090	49 239	33 648	1.46	....	0.011	0.146	....	0.009	0.196
494........	3.12	16	0.095	41 185	30 091	1.37	....	0.031	0.134	....	0.004	0.169
542........	2.38	14	0.090	29 413	19 575	1.50	....	0.058	0.178	....	0.008	0.244
547........	4.70	16	0.079	68 201	44 285	1.54	....	0.044	0.143	....	0.003	0.190
555........	3.08	14	0.110	31 343	25 368	1.24	....	0.047	0.145	....	0.008	0.200
769........	1.76	14	0.070	21 737	14 490	1.50	....	0.041	0.120	....	0.004	0.165
775........	4.42	14	0.090	52 512	36 302	1.45	....	0.037	0.115	....	0.008	0.190
776........	4.59	14	0.077	60 137	37 710	1.60	....	0.045	0.168	....	0.003	0.216
856........	1.13	16	0.095	14 440	10 581	1.77	....	0.022	0.171	....	0.007	0.200
Totals....	33.11	..	......	429 485	292 940	....	....	......	....	....	....	......
Averages.	3.31	15	$0.091	......	......	1.45	....	$0.040	$0.153	....	$0.006	$0.199
Residuum Products.—Application Made in Two Pourings.												
323........	2.44	16	$0.097	31 293	22 931	1.25	0.11	$0.063	$0.160	....	$0.011	$0.234
637........	3.54	14	0.094	45 064	29 063	1.30	0.25	0.067	0.155	....	0.008	0.231
789........	0.85	14	0.110	11 674	6 998	1.35	0.32	0.040	0.183	....	0.009	0.232
821........	4.07	14	0.095	60 607	33 415	1.33	0.49	0.075	0.178	....	0.009	0.262
5002........	1.69	16	0.095	23 353	15 111	1.10	0.44	0.023	0.160	....	0.003	0.186
5010........	1.98	16	0.090	32 718	18 638	1.41	0.34	0.033	0.191	....	0.008	0.232
5011........	1.73	16	0.096	28 748	16 259	1.30	0.47	0.080	0.160	....	0.015	0.265
5012........	2.78	16	0.093	43 041	26 079	1.25	0.40	0.035	0.220	....	0.004	0.259
5016........	2.97	16	0.093	48 289	27 806	1.29	0.44	0.056	0.176	....	0.006	0.238
5017........	2.46	16	0.078	37 608	23 141	1.20	0.43	0.064	0.172	....	0.008	0.244
Totals....	24.51	......	......	362 395	219 441	....	....	....	....	....	....	......
Averages.	2.45	15.4	$0.094	....	....	1.28	0.37	$0.054	$0.176	....	$0.008	$0.238

Mr. Armstrong. tabulations were made from a few of them, the highways selected being those rendering the best and most complete reports. The figures merely show the cost of completing the macadamized portion of the road, after the top stone has been placed and rolled, ready for the bituminous material and filler. As stated in Table 1, the average cost per square yard was

$0.199 for residuum products, one pouring;
0.238 " " " two pourings;
0.319 " fluxed Bermudez products, two pourings.

Though some of the figures in Table 1 are evidently in error, especially those showing prices of material, the errors are not large. In many cases they will compensate each other, and will not affect materially the average cost. Furthermore, as these figures represent the cost under somewhat varying conditions, for work done in different parts of the State, by different engineers and contractors, on approximately 70 miles of highway, it is believed that they are of more value, for determining the cost of roads of this type, than those obtained from a short-time observation on a single piece of work.

Mr. Thomes. E. H. Thomes, M. Am. Soc. C. E.—As the mineral aggregate constitutes about 90% of a bituminous pavement, it merits more consideration than it generally receives. There is considerable diversity of opinion among engineers as to the proper character and size of the mineral particles which should be used for paving purposes. Before any agreement can be reached it is necessary to be familiar with the materials available, the conditions attending their production, transportation and use, with the practical commercial limitations thereof, and also to designate the size by a uniform method, or to describe it definitely.

At present the size of the mineral matter is specified by the following methods: by the minimum and maximum dimensions of length; by average dimensions, or by diameter, which may mean several things; by passing through a ring or gauge, with or without stating the direction as every or one direction; by passing through or over a screen, with or without stating round or square opening; by passing screens with a definite number of openings or meshes, without stating the gauge of the wire; by passing over holes of a certain size in a revolving cylindrical screen and through holes of another size.

The size is designated, for instance, as ¾-in. stone, ¾ in. in size, ¾-in. gauge, or ¾ in. in diameter. Such a designation may mean the minimum, the maximum, or the average dimension, or the average diameter of the circular openings in the screens used, or it may refer to the diameter of the ring or gauge through which the stone must pass. Crushed stone passing through the ¾-in. circular perforations of a revolving stone screen will average smaller than that which will pass through a ring of ¾ in. internal diameter, as tested by hand, but it

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will be larger than the pieces which will pass a $\frac{3}{4}$-in. ring in every direction. In a revolving stone screen some pieces which would pass through a $\frac{3}{4}$-in. ring are carried over the moving $\frac{3}{4}$-in. perforation and deposited with stone of the next larger size. In the vicinity of New York City, commercial $\frac{3}{4}$-in. stone has been designated by one company as that passing over the $\frac{1}{2}$-in. circular perforations in a rotary screen and through the $1\frac{1}{4}$-in. openings, and another company designates $\frac{3}{4}$-in. stone as that passing over the $\frac{5}{8}$-in. perforations and through the $1\frac{1}{8}$-in. openings. Some designate the first course of bottom stone, which is usually the larger, as No. 1 stone, and some define the small stone which passes through the first perforated section of the screen as No. 1 stone. If stone is specified by dimensions, it is necessary to know the practical limits of production.

Crushed stone may be separated into sizes by fixed inclined screens with various kinds of openings, by shaker screens, or by other devices. Most of the crushed stone used, however, is separated by passing it through revolving cylindrical screens with circular perforations. As it becomes necessary to determine the dimensions of these perforations, it would seem best for the engineer to specify the size of the openings and designate the stone as 1-in. to 2-in. stone or gauge, referring to the diameters of the screen perforations. Unless both diameters of the openings are noted, the size of the stone is indeterminate. Due consideration must be given to the best utilization of all the product of the crusher, because, if only part of the product is used, the price increases. The engineer should inspect the stone plants in his vicinity to familiarize himself with conditions and with the commercial product obtainable. Most quarry men will welcome such inspection, as they are caused much trouble and expense by engineers not having a clear conception of the stone commercially obtainable under their specifications. Proper co-operation between quarry men and engineers will work to the benefit of all. When the engineer controls the entire output of a plant, he can secure any reasonable grading of stone necessary, by the proper length and perforations of the screens, etc.; but, where stone is supplied to a number of consumers from one large plant, it is readily seen that the machinery cannot be changed to suit the whims of each. Special requirements or restrictions and disregard of practical commercial conditions will unnecessarily increase the cost. It becomes necessary to produce a few stock sizes which will best satisfy the demand for various purposes. If the engineers of this Society will agree upon a practicable designation of the sizes of crushed stone and determine standard sizes which will comply with the best commercial requirements, much trouble and expense can be saved to all parties.

The quantity of dust in stone will depend on the character of the rock, the care in stripping and handling, the quantity of moisture

Mr. Thomes. present (as more dust will adhere to stone in wet weather), the velocity and direction of the wind, the type of crusher, the size, length, and condition of the screens, perforations, etc., the distance the stone falls, and the methods used in transportation, construction, rolling, etc. A certain quantity of stone dust may be of advantage in stone for water-bound macadam, or for concrete, if it is uniformly distributed, but, in bituminous pavements, the question of dust and dirt is more important. It is generally specified that the stone for bituminous pavement shall be perfectly dry and free from dust. This is the ideal condition, but it can hardly be attained in practice. As coal-tar products usually penetrate and adhere to dusty stone more readily than asphaltic products, less dust should be permitted with the latter. The stone may be washed by a water sprinkler after it is rolled in place, but it is a question as to what degree this is advisable for bituminous work. Under some conditions, absence of dust is more important than absence of moisture. Dust and dirt should be defined, and the percentage by weight passing a screen with openings, say, $\frac{1}{100}$ in. square, should be specified.

Stone screens are usually from 3 to 5 ft. in diameter, and from 8 to 30 ft. long. They are set up at an inclination of about 1 to 1½ in. per ft., and operated at a peripheral speed of about 200 ft. per sec. The slower the speed and the flatter the inclination the better the separation and the less the dust, but the output is decreased. The longer the screen the better the separation, but there is more dust and more rounding of the stone. The crushed stone usually passes the dust jacket and from the small holes to the larger ones, and the tailings, rejected at the end of the screen, are re-crushed. All the stone may first be passed through a preliminary scalping screen and then dropped to a final sizing or separating screen. If the stones were passed through separate screens, the large ones could be taken out separately, being removed by the last screen, but such restrictions would increase the cost, and it is necessary to determine the degree of screening advisable.

It is commonly specified that crushed stone shall be cubical in shape. This may be the ideal condition, but it can seldom, if ever, be obtained in practice, especially with trap rock, which is considered the best for road purposes, as it has no grain, rift, seam, or regular place of fracture, like some rocks, and breaks into shape more like a buckwheat grain, with no two sides parallel or regular. The fracture or shape cannot well be specified, as it depends mostly on the character of the rock and to some extent on the methods used, and the size, type, and character of the crusher. The gyratory crusher may produce a more uniform size and shape than the jaw type. Hand-broken and sorted stone is better than a machine product, but is out of the question.

Crushed quarry stone is more uniform and better than field stone, but some field stone is very durable. Stone from the same quarry and apparently of the same texture may vary somewhat in wearing qualities. The durability of, and general satisfaction with, a bituminous pavement depends as much on the character and grading of the mineral aggregate as on the character of the bituminous cement. In some respects, the character of the mineral matter is less important in bituminous than in water-bound macadam. The stone in the latter shows rounding of edges, wear, and movement to some depth below the surface, whereas in bituminous macadam this is not so apparent. Mr. Thomes.

The following is the average of three tests of Hudson River trap rock made by the United States Office of Public Roads:

Name, Diabase (trap rock). Character of material, igneous rock.

Specific gravity	2.95
Weight, in pounds per cubic foot	184
Water absorbed, in pounds per cubic foot	0.30
Percentage of wear	2.5
French coefficient of wear	16.0
Hardness	18.3
Toughness	26
Cementing value	good.

While service tests are the only positive indication of the wearing qualities of a rock, laboratory tests are a valuable guide, and if engineers will have their road metal tested, and will record properly all the local conditions and results, the value and reliability of these tests will increase. The resistance to wear, or the abrasion test, and the toughness, or impact test, are the more important ones, though specific gravity, hardness, absorption, cementing value, crushing strength, and chemical analysis may be of value when considered with the other conditions. Instead of stating indefinitely that the rock shall be hard, tough, and durable, it would be better to specify definite practicable limits, in accordance with the foregoing tests.

The majority of engineers seem to prefer about 1 to 1½-in. stone for the upper course, but some claim that the 2 to 2½-in. stone is better, especially for heavy traffic. A few contend that the 1 to 3-in. gauge is the best size. Theoretically, the last may be the best, because this range of sizes has less total voids if the stones of various sizes can be distributed uniformly in the pavement, but, with a wide range in the sizes of screen perforations, it is practically impossible to prevent the large and small stone from segregating when deposited on the road. The large stone will fall to the outside of the pile and the fine fragments will remain in the center, and this will cause unequal wear and holes in the pavement. With a soft stone, a large

Mr. Thomes. size may give the best results. Stones of large size may make a more durable road if it is properly constructed, but more care is required in thoroughly packing and rolling. More bituminous binder may be required unless stone chips, sand, or screenings are used to fill the voids partly if a large-sized stone is used, and it is difficult to distribute such materials uniformly. A pavement constructed of large stone is more likely to loosen, pick up, and ravel, it has a rougher and more open surface, wears unevenly, and is difficult to repair. Small stone costs more to crush, and contains more dust and waste, but it is more easily handled, produces a smoother and closer surface, wears more uniformly, requires less binder, and is more readily repaired. Small stone is better for patching. The maximum size of the stone may be about one-half the depth of the stone layer.

Stone may be purchased by the cubic yard or by the ton; if by the cubic yard, a more uniform and cleaner stone is apt to be obtained, as the seller secures a larger volume when the product is separated into a number of uniform sizes. If bought by weight, the stone is more likely to contain dust and dirt. It should be definitely stated, when, where, how, and by whom, the stone shall be measured or weighed, whether at the plant or at point of delivery, whether loose or settled, whether on boat, car, truck, etc., and as to what allowance shall be made for dust, moisture, etc. Screenings will weigh more per cubic yard when dry than when damp, because in the latter condition they swell. The following are approximate weights, in pounds per cubic yard, of Hudson River trap rock measured on scows:

Solid rock	4 970
Run of crusher	2 850
$2\frac{1}{4}$ to $3\frac{1}{4}$-in. perforation, or commercially called $2\frac{1}{2}$-in. stone	2 600
$1\frac{1}{8}$ to $2\frac{1}{4}$-in. perforation, or commercial $1\frac{1}{2}$-in. stone	2 475
$\frac{5}{8}$ to $1\frac{1}{8}$-in. perforation, called $\frac{3}{4}$-in. stone	2 400
Passing a $\frac{5}{8}$-in. perforation, called screenings	2 650

The following specifications for size of stone are submitted for suggestions and criticisms, and as a basis for further discussion:

The product of the rock crusher shall be separated into four grades or sizes by an approved method which will produce stone equal to that obtained in the best commercial practice. The separation shall be done in a rotary stone screen having circular perforations of the following diameters: $\frac{1}{2}$ in., 1 in., 2 in., and 3 in. These grades shall be designated as: screenings, $\frac{1}{2}$ to 1-in. stone, 1 to 2-in. stone, and 2 to 3-in. stone, respectively. Screenings which contain less than 5% by weight of dust passing an opening $\frac{1}{100}$ in. square shall be designated as stone chips. The engineer may permit a slight variation in the sizes, to suit the commercial materials obtainable. For bituminous pavements built by the penetration method, the 2 to 3-in. stone would be laid in the lower course and chips would be used to fill the voids. The 1 to 2-in. stone would be spread next, followed by the bituminous binder, then the $\frac{1}{2}$ to 1-in. stone and seal coat, which would be covered with chips. For the mixing method, any combination or percentages may be used. Mr. Thomes.

In order that a clear understanding may be had when the size of the mineral aggregate is referred to, it is requested that engineers indicate definitely how the material is actually produced, by stating the dimensions of the screen sections and perforations, or otherwise.

The experimental pavements constructed in the summer of 1911 in the Borough of Queens, New York City, on Hillside Avenue, Jamaica, extend eastward from the New York and Queens County Street Railway for a distance of 2 000 ft. to the Soldiers' Monument at Bergen Avenue. The first 200 ft. was constructed with oil-cement-concrete 4 in. thick. The 1:2:4 concrete was mixed with mineral oil to the extent of 10% of the weight of the cement. The concrete was covered with about $\frac{1}{2}$ in. of lean oil-cement-mortar. A length of 1 200 ft. was built of bituminous concrete in proportions of 18 gal. of bituminous material to 1 cu. yd. of commercial $\frac{3}{4}$-in. stone, of $\frac{1}{2}$ to $1\frac{1}{4}$-in. gauge, laid and rolled to a compacted depth of 2 in. A $\frac{3}{4}$-gal. seal coat was applied to this surface and covered with stone chips. A length of 600 ft. was built of bituminous macadam by the penetration method. A 3-in. layer of loose stone was treated with $1\frac{1}{2}$ gal. of binder, followed by a coating of stone chips. A $\frac{3}{4}$-gal. seal coat was then applied and covered with stone chips. The foundation was an old macadam road which was scarified and brought up to grade with a $1\frac{1}{2}$-in. layer of new stone. The materials for the first 1 400 ft. were mixed in a Smith hot mixer, No. 11. The bituminous materials were heated in a 375-gal., Stevenson and Leonard, heating kettle, and were applied by a Good Roads distributor and Perfection hand-pouring pots. The speaker was in charge of the work for the Borough of Queens, which furnished the labor, materials, and equipment. The work was constructed under the general direction of a Committee of which Nelson P. Lewis, M. Am. Soc. C. E., is Chairman, and in co-operation with the U. S. Office of Public Roads. A census showed an average daily traffic of 1 600 vehicles, mostly motor vehicles.

The work is divided into the following sections:

Section 1, Station 0 (at railroad) to Station 0 + 97, Standard Oil cement concrete.

Section 2, Station 0 + 97 to 1 + 73, Texas Oil cement concrete. Station 1 + 73 to 1 + 97, old stone and brick pavement left in for a cross-gutter.

Section 3, Station 1 + 97 to 4 + 96, Texaco Macadam Binder, a

Mr. Thomes. cut-back oil asphalt, in mix, and Texaco Asphalt 55 special cement for a seal coat.

Section 4, Station 4 + 96 to 8 + 00, Bermudez Road Asphalt, a fluxed native asphalt, in mix and seal coat.

Section 5, Station 8 + 00 to 9 + 50, Tarvia X, a heavy refined coal-tar, in mix and seal coat.

Section 6, Station 9 + 50 to 10 + 98, Tarvia X, in mix, and Texaco Asphalt 55 Special for seal coat.

Section 7, Station 10 + 98 to 13 + 94, Standard Special Binder, a cut-back oil asphalt, in mix and seal coat.

Section 8, Station 13 + 94 to 14 + 75, penetration method, Texaco Road Asphalt first application, and Texaco Asphalt 55 for seal coat.

Section 9, Station 14 + 75 to 15 + 50, penetration method, Texaco Road Asphalt for both applications.

Section 10, Station 15 + 50 to 17 + 00, penetration method, Tarvia X for both applications.

Section 11, Station 17 + 00 to 18 + 50, penetration method, Bermudez Road Asphalt for both applications.

Section 12, Station 18 + 50 to 20 + 00, penetration method, Standard Binder B for both applications.

For a distance of about 50 ft. east of Station 20 the Bureau of Highways laid a sample of Amiesite pavement for which the materials were donated by the Long Island Amiesite Company.

Mr. Pillsbury. F. C. Pillsbury, M. Am. Soc. C. E.—During the past season, greatly increased knowledge of all bituminous materials and how to use them has been gained. One point, which has been brought out more clearly than others, is the necessity for experienced men on the work where the actual handling of the materials is going on, and the payment of wages and salaries sufficient to provide for the continuous employment of such men, so that they may remain on the work long enough to make real advances. The speaker believes that the use of bituminous or other materials requires especially intelligent supervision and labor, in order to obtain satisfactory results, that such results can be obtained in no other way, and that it takes time to develop the necessary knowledge and skill. Most of this work is done under the direction of State, county, or municipal employees, and this point is mentioned because it is so frequently the custom, in public service, to change these employees, on whom so much depends. The speaker has never heard any one disagree with the statement that the personal factor is of prime importance. If, among engineers, there seems to be an apparent lack of interest where this matter is concerned, it can only be ascribed to professional modesty, and if this modesty prevents an engineer from acting in his own behalf, need it prevent some kind of united action by such a body as this Society, which certainly stands for the highest principles, and to which engineers all over the world look for standards in all engineering matters? Mr. Pillsbury.

Engineers have not yet been able to arrive at an agreement on uniform tests to apply to oils, tars, and asphalts. Certainly, no tests have been furnished which can be used in the field. Usually, it is necessary to depend on the word of the dealer, as to the nature of the material furnished. Tests are made by various departments, but, usually, the results have not become known to those actually directing the work until after the material has been used or partly used. There is a great need for the development of some simple tests, born of a practical knowledge of what certain materials will do, and our engineers and inspectors must be educated in some way to that extent. So little is known in general about the various materials, that tars and asphalts are frequently spoken of as oils, and *vice versa,* by men who have used great quantities of them. Even some of the dealers have trade names for certain materials in certain localities and other names for the same materials at other places. One of the greatest things that engineers can do is to work toward the standardization of bituminous materials and methods, and this matter, the speaker believes, has already been taken up by the Special Committee on Bituminous Materials for Road Construction.

Much advancement has been made in the application and use of bituminous binders, and this has led to a better knowledge of the types of surfaces which should be built; therefore, it seems that there has been a gradual elimination of many of the features and methods, as well as materials, which have been subjects for discussion in the past. It is not practical, although it is possible, to build a great many different types of road which will answer certain purposes, and to use many different kinds of materials, which would probably give approximately the same results, but would require a greatly varied knowledge in handling. In the near future, it is probable that, for the average country highway, the types of road to be constructed will be few in number, and will then depend more on the mineral aggregate available than on anything else, excepting, of course, the weight and volume of traffic.

One type of road which has been developed under the speaker's observation is a bituminous macadam constructed as follows:

First: Assuming that the sub-grade and surface are properly drained, on the foundation is first placed a layer (4 in. after rolling) of egg-size broken stone, $1\frac{1}{4}$ to $2\frac{1}{2}$ in. in longest dimension. This layer is thoroughly bound with stone dust or other suitable material, rolled, and flushed with water until it is practically impervious to the bituminous material.

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Second: On this heavy asphaltic oil is evenly distributed, by a pressure distributor, $\frac{3}{4}$ gal. per sq. yd.

Third: On the oil a layer of nut-size broken stone, $\frac{1}{2}$ to $1\frac{1}{4}$ in. in longest dimension, which will roll to 2 in. thick, is immediately placed and carefully spread with shovels, the carts containing it being driven along the side of the road. The depth of the stone is regulated by wooden cubes placed on the first course, which is sanded at the points where the cubes are placed, in order to prevent them from sticking to the oil. This nut-size stone is then compacted with a steam roller.

Fourth: On the nut-size stone is distributed under pressure about $\frac{5}{8}$ gal. of the same kind of oil as used before, the application being absolutely perfect in distribution, and the penetration reaching well down into the stone.

Fifth: On the oil is immediately spread fine gravel, gravelly sand, or stone screenings, just sufficient in quantity to fill the surface voids in the nut-size stone, and to take up the thin coating of oil left on the top. This is then thoroughly watered and compacted by the steam roller until there are no signs of movement.

The actual costs of such work to the contractor, on two State roads in Massachusetts built in 1911, has been furnished to the speaker by Mr. D. H. Dickinson, the Resident Engineer, and are as shown in Tables 2 and 3.

The traffic on these roads does not consist of a large volume of heavy horse-drawn vehicles, but probably from 500 to 600 automobiles per day pass over them, except during the winter, and there is also considerable heavy farming and other teaming.

In 1908 a number of experiments in mixing sand, gravel, and broken stone with bituminous materials were conducted. These were continued in 1909, and were carried out with oil and gravel to such an extent that it was possible to make definite comparisons while the work was in progress, and since it has been under traffic. These experiments and the later observations warrant specifications by which such work can be described when the conditions and materials available make its use advisable. Realizing the variation in the gravel which occurs even on any particular piece of work, the speaker has provided opportunity for changing the proportions; when the proportion of sand increases, there should be an increase in the quantity of bituminous material, and *vice versa*. For other reasons which may develop, the quantities may need to be varied in order to obtain correct results. This bituminous mixture may be used in re-surfacing old macadam or old gravel roads, or on gravel, macadam, or other bases in new work, and by improving the bituminous material, comparatively heavy traffic may be sustained. All these variations should be based on experience, if possible. The specifications follow:

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TABLE 2.—ACTUAL COSTS OF BITUMINOUS MACADAM CONSTRUCTED AT NORTH ANDOVER, IN 1911.

Length, 8 107 ft.; area, 13 615 sq. yd.; width, 15 ft.

Broken Stone:			
Bottom course:			
Stone. 2 898 tons, at $1.15		$3 332.70	
Cost of laying, including unloading, teaming, spreading, rolling, binding, etc.		1 637.70	
Total cost for bottom course		$4 970.40	
Cost per square yard	$0.3651		
Broken stone dust used to bind bottom course.			
Top Course:			
Stone, 1 148 tons, at $1.15		$1 320.20	
Cost of laying		686.21	
Total cost for top course		$2 006.41	
Cost per square yard	$0.1473		
Oiling:			
First application, ¾ gal. per sq. yd.			
Cost of oil per square yard	$0.0438		
" " heating per square yard	0.0225		
" " hauling " " "	0.0025		
" " applying " " "	0.0168		
Total cost per square yard	$0.0856		
Second application, ⅝ gal. per sq. yd.			
Cost of oil per square yard	$0.0369		
" " heating per square yard	0.0192		
" " hauling " " "	0.0022		
" " applying " " "	0.0168		
Total cost per square yard	$0.0751		
Sand covering:			
Total cost per square yard	$0.0373		
Cost per square yard for bottom course, broken stone		$0.3651	
" " " " " top " " "		0.1473	
Total			$0.5124
Cost per square yard for first application of oil		$0.0856	
" " " " " second " " "		0.0751	
			0.1607
" " " " " sanding			0.0373
Total			$0.1980
Total cost per square yard for pavement			0.7104

Average length of haul for broken stone, 1 mile.
" " " " " oil, 1 "
" " " " " sand covering 1 " ; sand not screened.
Broken stone trap rock shipped by rail.
Oil delivered in tank cars.
Cost of labor, 22 cents per hour.
" " teams, 55 " " "

BITUMINOUS SURFACING.

On the sub-grade and foundation shall be spread, according to lines and grades to be given by the engineer, the bituminous mixture which will form the wearing surface. It shall consist of sand and gravel or broken stone mixed with asphaltic oil. The sizes of stone and sand, the proportions of the same and of oil, and the method of mixing and placing shall be as described hereinafter.

Thickness.—The bituminous surfacing shall be laid in one course, and, after rolling, shall be 2 in. thick at the center and 2 in. thick at the edges.

Mr. Pillsbury.

TABLE 3.—ACTUAL COSTS OF BITUMINOUS MACADAM CONSTRUCTED AT TYNGSBORO, IN 1911.

Length, 4 350 ft.; area, 7 250 sq. yd.; width, 15 ft.

Broken stone:		
Bottom course:		
Stone, 1 492.05 tons, at $1.15		$1 715.86
Cost of laying, including unloading, teaming, spreading, rolling, binding, etc.		1 137.00
Total cost for bottom course		$2 852.86
Cost per square yard	$0.3935	
Broken stone dust used to bind bottom course.		
Top course:		
Stone, 612.49 tons, at $1.15		$704.36
Cost of laying		439.50
Total cost for top course		$1 143.86
Cost per square yard	$0.1579	
Oiling:		
First application, ¾ gal. per sq. yd.:		
Cost of oil per square yard	$0.0438	
" " heating " "	0.0301	
" " hauling " "	0.0026	
" " applying " "	0.0160	
Total cost per square yard	$0.0925	
Second application, ⅝ gal. per sq. yd.:		
Cost of oil per square yard	$0.0369	
" " heating " "	0.0252	
" " hauling " "	0.0024	
" " applying " "	0.0160	
Total cost per square yard	$0.0805	
Screened gravel covering:		
Total cost per square yard	$0.0589	
Cost per square yard for bottom course, broken stone	0.3935	
" " " " " top " " "	0.1579	
Total		$0.5514
Cost per square yard for first application of oil	$0.0925	
" " " " " second " " "	0.0805	
" " " " " graveling		$0.1730
		0.0589
Total		$0.2319
Total cost per square yard for pavement		0.7833

Average length of haul for broken stone, ¾ mile.
" " " " " oil ¾ "
" " " " " screened gravel covering, about 1 mile.
Broken stone trap rock shipped by rail.
Oil delivered in tank cars.
Cost of labor, 22 cents per hour.
" " teams, 55 " " "

Width.—The width shall be..................................

Sand.—The sand shall consist of particles which will pass through a screen with meshes ¼ in. square. It shall contain not more than 5% of clay or loam, and be free from adventitious matter.

Stones.—The gravel or broken stones shall run in size from ¼ to ¾ in., and stones larger than ¾ in. in their longest diameters shall not be used.

Proportions of Stones and Sand.—Not more than 75% nor less than 15% of the mineral aggregate shall consist of stones of sizes as hereinbefore specified, and the proportion of sand as hereinbefore specified shall vary according to the proportion of stone used. Mr. Pillsbury.

Oil.—The bituminous material shall be according to the specifications for the same.

Heating Stones and Sand.—Mixing.—When the sand and stones have been heated to not less than 180° Fahr., or more, if the Engineer requires it, they shall be mixed with the oil, either by hand or by machinery, and as the Engineer may direct, until all particles of sand and stone are covered with oil.

Before mixing, the stone and sand shall be heated separately, and carefully measured to obtain the correct proportions.

Proportions of Oil and Mineral Aggregate.—Not less than 15 gal. nor more than 20 gal. of oil, as the Engineer may direct, shall be mixed with each cubic yard of gravel or stone and sand.

Heating Oil.—Before mixing with the sand, the oil shall be carefully heated to not less than 200° Fahr., and, at that or such higher temperature as the Engineer may direct, shall be mixed with the sand and stone. No oil shall be used after it has been injured by overheating or burning. The Contractor shall heat the oil in suitable kettles or by steam coils, or in such manner as may be satisfactory to the Engineer.

Spreading.—After being properly prepared, as hereinbefore specified, the mixture shall be hauled to the road and spread before it has cooled to a temperature of less than 100° Fahr.

The mixture shall be dumped on steel dumping platforms, or shoveled directly from the cart into place. As the spreading is done, rakes shall be used to obtain a uniform distribution of stones and sand and an even surface before rolling.

Temperature at Spreading.—No oil and gravel shall be mixed or spread when the temperature of the atmosphere is below 50° Fahr., and not any of this work shall be done after September 30th.

Rolling.—The material, after being spread and raked satisfactorily, shall be at once rolled with a steam roller, care being taken not to push the mixture out of place, but to roll so as to lay it down, compressed to a perfect cross-section, and true to line and grade. During very hot weather the rolling shall be done at night or early in the morning, or postponed until cool enough to roll without pushing out of place and shape.

Time to Elapse Before Use.—No teaming or travel of any kind shall be allowed to pass over the new surface until 24 hours have elapsed after the final rolling, or until the surface has become sufficiently hardened not to be injured by picking up or tracking.

ARTHUR H. BLANCHARD, M. AM. SOC. C. E.—The introduction during 1911 of low first-cost mixing machines, equipped with suitable attachments for heating the aggregate, will be the cause, without Mr. Blanchard.

Mr. Blanchard. doubt, of the adoption of the mixing method by many engineers, and of a relative decrease in the use of the penetration method. The time is opportune for the introduction of mixing machines, as the fact is becoming recognized that the average quality of bituminous pavements constructed by the mixing method is decidedly above that of bituminous pavements constructed by penetration methods. This observation pertains to types of bituminous concrete pavements and bituminous macadam pavements in which one-size, crusher-run stone is used in the top or wearing surface.

Bituminous concrete pavements in which broken stone is used may be classified under three heads, dependent on the character of the mineral aggregate:

First.—Aggregates composed of one-size, crusher-run stone;

Second.—Aggregates of one-size, crusher-run stone and sand;

Third.—Finely graded aggregates of stone and sand, with or without the addition of fine mineral matter.

Machines may be purchased for less than $2 000 which will coat satisfactorily an aggregate of the first class with all the different kinds of bituminous materials used during 1911 in work of this class. Although certain aggregates of the second class have been mixed satisfactorily with these machines, the speaker does not wish to make a general statement covering the mixing of all aggregates of this class. More expensive machines have been manufactured, which are economical for mixing aggregates of the second and third types. With efficient machines, the total cost of labor for mixing a 2-in. rolled wearing course should not exceed from 3 to 6 cents per sq. yd.

In many instances fear of injunctions and lawsuits has been responsible for the non-adoption of the mixing method; and it is of interest to note that certain patentees have admitted in writing that the construction of a certain type of bituminous concrete pavement of the first class, composed of one-size, crusher-run stone, is not an infringement of their patent No. 727 505.

In Washington, during 1910 and 1911, there have been constructed, under the direction of the Engineer-Commissioner of the District of Columbia, bituminous concrete pavements having aggregates of the second class. The specification descriptive of the mineral aggregate is: "Trap rock screenings, 2 parts; concrete sand, 1 part; and mineral dust, at least 5% of the above aggregate."

In the cases of Topeka, Kans., and Creston, Iowa, decrees were entered which were agreed to by all interested parties, including the patentees previously referred to, that a certain type of bituminous concrete pavement of the third class is not covered by U. S. Patent No. 727 505. The final decree stated that the composition referred to consisted of stone passing a ½-in. ring, and that less than 10% of the stone and sand should be retained on a screen with openings ¼ in.

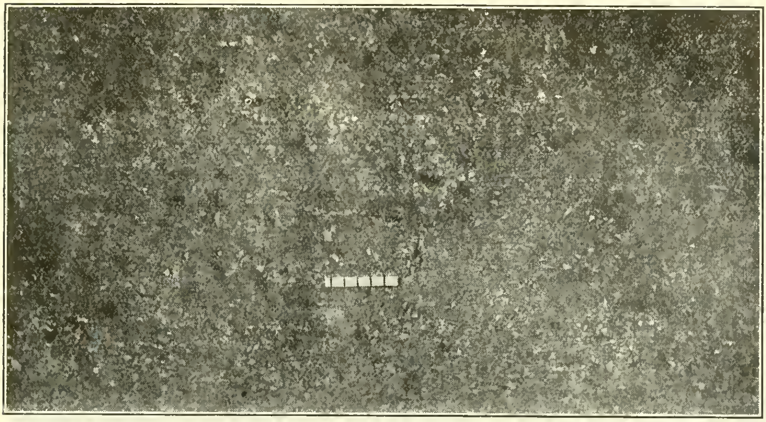

FIG. 3.—SURFACE OF UGITE SECTION.

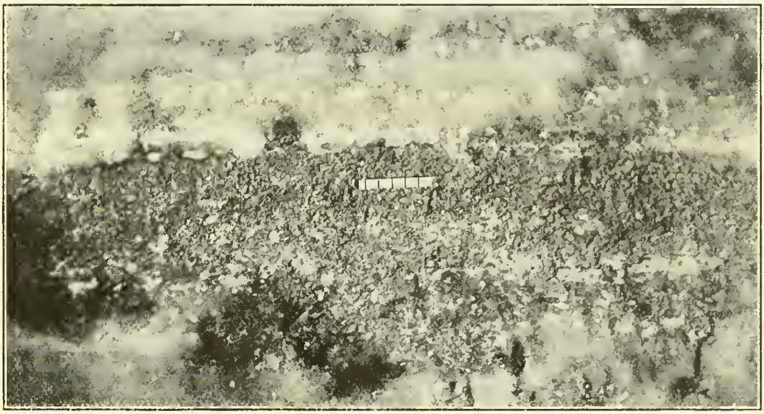

FIG. 4.—SURFACE OF TARITE, MALDEN SECTION.

FIG. 5.—SURFACE OF TARITE, ASPHALT 10 PER CENT. SECTION.

FIG. 6.—SURFACE OF TARITE, ASPHALT 20 PER CENT. SECTION.

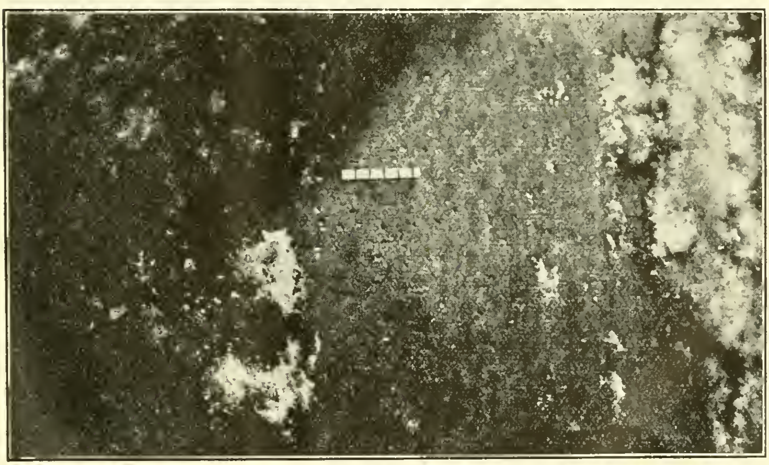

FIG. 7.—SURFACE OF TEXACO MACADAM BINDER SECTION.

Mr. Blanchard.

in diameter, and that the pavement could be constructed by the following formula:

Bitumen	from	7 to	11%
Mineral aggregate passing 200-mesh screen,	"	5 "	11 "
" 40 " "	"	18 "	30 "
" 10 " "	"	25 "	55 "
" 8 " "	"	8 "	22 "
" 2 " "	less	than	10 "

The speaker has given* detailed descriptions and cost data covering the construction of a series of experimental sections of bituminous concrete pavements which were laid in the fall of 1909 on a section of a State Road in the Town of Barrington, Rhode Island, under his direction while he was Deputy Engineer of the State Board of Public Roads. The condition of the surfaces of the various sections in December, 1910, was also described.

A few facts relative to the condition of these various sections, as observed in January, 1912, will now be presented. It should be noted that all these bituminous concrete pavements were constructed with the same labor, under the same supervision, and by using the same kind of mineral aggregate. The road is at present subjected to a mixed traffic of about 100 horse-drawn vehicles and 250 to 300 motor cars per day. Many of the motor vehicles are of the large touring-car type, and travel at high speeds. The experimental sections were constructed to determine the most economical and satisfactory bituminous material to be used both as the cement and for the seal coat for bituminous concrete pavements subjected to trunk highway traffic. Up to January, 1912, no repairs have been considered necessary, and no new material of any kind has been applied to the surface.

The photographs of the surfaces of the various sections show certain characteristics which will be referred to later. It will be noted that a section of a rule 5½ in. in length was photographed on each surface, thus giving some idea of the size of the stones in the surface.

The section built with refined water-gas tar, Ugite No. 4, presented a mosaic surface, well bonded and smooth, which was not slippery, although slipperiness had been characteristic of this section during the winter of 1910-11. Apparently, it will need no repairs during 1912.

The section constructed with a combination of refined coal-tar and 10% asphalt, Tarite-Asphalt 10%, should be treated with a superficial coat of some kind of bituminous material. The surface presented a distinct open mosaic appearance. This surface has not been slippery since it was built.

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* *Transactions*, Am. Soc. C. E., Vol. LXXIII, p. 100.

Mr. Blanchard. The section built of a combination of refined tar and 20% asphalt, Tarite-Asphalt 20%, had an appearance somewhat similar to that of the section built with refined water-gas tar, and should not require any maintenance during 1912, but will need a superficial coat of bituminous material in 1913. This surface has not been slippery since it was built.

Both sections built of refined coal-tars, Tarite-Malden and Tarite-Springfield, exhibited an open surface, which, although providing an excellent foothold, will probably disintegrate under a horse-drawn vehicle traffic of 100 to 200 vehicles per day unless properly maintained. Of course, a bituminous surface of tar would increase its life and would be efficacious for from 1 to 2 years, dependent on the development of traffic. Either a tar-asphalt compound or an asphalt would be more effective, and would, without doubt, provide a satisfactory surface for at least 3 years.

The section constructed with Texaco Macadam Binder had a surface similar in appearance to that of a sheet-asphalt pavement. This surface is not slippery. The surface was slightly indented with the horses' calks even with the air temperature at 12° Fahr. Apparently, it will need no maintenance for 2 or 3 years.

It is of interest to note that adjacent sections of the standard type of bituminous concrete pavement adopted for the Rhode Island State Roads in 1909, that is, coal-tar for the mix and a Texaco asphalt, solid at air temperature, for the seal coat, were in excellent condition. They have had no repairs, and apparently will need none during the season of 1912. A photograph of this surface appears similar to that shown for the section where Texaco Macadam Binder was used in the mix and for the seal coat.

Mr. Crosby. W. W. Crosby, M. Am. Soc. C. E.—There has been some question as to the relative values of the different methods, and there seems to be a preponderance of argument in favor of the mixing method and against the penetration method. It seems to the speaker that each engineer will have to be guided in each case by the particular conditions as to which method will be most valuable.

The speaker has never found it necessary to use a seal coat in the penetration work done under his direction. In fact, there have been a number of instances where the absence of a seal coat has been of perceptible advantage, and where there has not yet been any apparent disintegration whatsoever. This result has been secured even when as little as $1\frac{1}{4}$ gal. per sq. yd. of bituminous material has been used. The method of constructing the surface course seems to make no difference as to its need for a seal coat. For instance, in Maryland, the second course may be thoroughly rolled, filled with sand or stone chips, penetrated with the bituminous material, and covered with chips, or it may be built by rolling the second course thoroughly, and applying the bituminous material before putting on any chips

Mr. Crosby.

whatever. All penetrated surfaces, whatever material is used, have the tendency to seal themselves eventually. Sometimes the process may be very slow, while at other times it is quite rapid, practically regardless of the quantity of bituminous material used. It has been noticed that where the surface was not filled with chips before the application of the bituminous material, it generally takes longer to acquire a smooth surface than where the chips were used—about a year, perhaps, in the first case, and from 2 to 3 months or less in the second. There is no perceptible difference in the surfaces, however, as soon as a smooth condition is obtained.

For the purpose of continuing, for the benefit of those interested, the previously recorded information concerning the work on Park Heights Avenue, Baltimore, Md.,* the writer submits Table 4, which is believed to be largely self-explanatory, and includes the expense on this road to January 1st, 1912.

Attention is called to the fact that Sections 15, 16, 28, and 29, are really surface treatments, and are improperly included perhaps under this topic. On the other hand, to report on them separately might be more objectionable than to include them in this discussion, and, therefore, the speaker hopes that there will be no objection to them, under the circumstances.

Table 5 shows the results of traffic censuses taken at various dates in 1910 and 1911.

Sections 1 and 3 were resurfaced in (August to October, inclusive) 1911, because of their uneven and unsatisfactory surface conditions. The older surface was scarified to a depth of about 3 in., and the mixture of stone and pitch removed. Clean No. 2 trap rock was then spread to a depth of 4 in. (loose), rolled, and poured with 1½ gal. per sq. yd. The stone was then chipped, and a flush coat of ½ gal. per sq. yd. was applied, a final coat of chips being spread on the latter. Section 2 remains somewhat wavy, as was the case a year ago.

Complaints were made that Sections 5, 7, 19, 20, and 23, were slippery in cold weather, and the expense reported on these sections this year is largely that of attempting to overcome this slipperiness. The latter is, in the speaker's judgment, due more to the excess of the pitch originally used, to the grades, and, perhaps, in some cases, to the high crown existing on these sections, than to other factors.

Sections 6, 8, 23, 24, 25, and 26, required the rolling in of additional stone, in order to absorb the excess of pitch appearing in hot weather.

Section 12 bled somewhat during warm weather, and the expense reported on it is largely for overcoming the resulting condition.

Section 15 required patching in numerous small spots (less than 1 sq. ft. in area) during 1911, and the expense therefor is noted. It

* *Transactions*, Am. Soc. C. E., Vol. LXXIII, p. 74.

Mr. Crosby.

TABLE 4.—BITUMINOUS WORK ON

Sect.	Material.	Survey Stations. From:	Survey Stations. To:	Linear feet.	Width, feet.	Locations. Street.	Locations. Station.
1..	Texas........................	0 — 350	37 + 50	320 65 3 715	30 35 24	Wiley Av. Belvidere Av.	0 37 + 50
2	Gulf Ref. Co.—Asphaltoil. A........................	37 + 50	44 + 76	726	24	....	....
3..	Texas........................	44 + 76	49 + 42	466	24	Hayward Av.	46 +
4..	Imp. Prod. Co.—Fairfield....	49 + 42	53 + 82	440	24	....	....
5..	U. G. I. No. 4—Sample No. 1	53 + 82	60 + 24	642	24	....	....
6..	Warren-Puritan Brand No. 10	60 + 24	67 + 09	685	24	Rogers Av.	60 + 75
7..	Tarvia X......................	67 + 09	74 + 25	716	24	....	....
8..	Amer. Tar Co.—Tarite.......	74 + 25	81 + 06	681	24	Glen Av.	81 +
9..	U. G. I. (1909 Work)..........	81 + 06	85 + 45	439	24	....	....
10..	U. G. I. (1910 Work)..........	85 + 45	97 + 57	1 212	24	Taney Rd.	96 +
11..	Texas........................	97 + 57	120 + 00	2 243	24	....	....
11 A	Mixed—52 Barrels............	120 + 00	124 + 73	473	24	Clark's Lane	121 +
12..	Headley Manf. Co............	124 + 73	142 + 30	1 757	24	Belmont Av.	138 +
13..	Barber Asphalt..............	142 + 30	158 + 06	1 576	24	Seven Mi. L.	150 +
14..	Fairfield—Tar................	158 + 06	170 + 00	1 194	24	Slade Av.	170 +
15..	Fairfield—Antidust..........	170 + 00	199 + 04	2 904	18	Opposite Suburb. Club.	180 +
16..	U. G. I.—Antidust...........	199 + 04	204 + 54	550	18	Court Rd.	204 +
17..	Sarco.........................	204 + 54	231 + 30	2 676	12	South of Barracks	231 +
18..	Std. Oil—(1910 Work)........	231 + 30	245 + 50	1 420	12	Barracks	about 240 +
19..	Std. Oil—(1909 Work)........	245 + 50	246 + 32	82	14	....	....
20..	U. G. I........................	246 + 32	254 + 64	832	14	....	....
21..	Flannigan Texas............	254 + 64	259 + 10	446	14	Hooks Lane.	259 +
22..	Gulf Asphaltoil.—A..........	259 + 10	265 + 38	628	14	....	....
23..	Warren-Puritan Brand No. 17	265 + 38	270 + 81	543	14	Ice Pond.	271 +
24..	U. G. I........................	270 + 81	285 + 25	1 444	14	Stone Arch Culvert.	285 + 90
25..	Imp. Prod. Co.—Fairfield....	285 + 25	295 + 10	985	14	B. Keyser's Entrance.	294
26	Con. G. E. L. & P. Co. complete top of U. G. I...	295 + 10	319 + 26	2 416	14	R. R. Bridge.	305 + 25
27	Con. G. E. L. & P. Co......	319 + 26 325 + 72.5	325 + 72.5 326 + 00	846.5 27.5	14 12	S. Side Valley Rd.	326 + 0
28..	Texaco Special..............	326 + 10	331 + 65	555	12	N. Side Valley Rd.	326 + 10
29..	Glutrin.......................	331 + 65	431 + 92	10 027	12	Coves Rd.	431 + 92

Length, 8.246 + miles.

is now in first-class condition, and, apparently, is likely to remain so, for the present, at least.

Section 16 appears to be losing life, and will probably require extensive re-treatment in 1912.

Section 17 showed a perfect mosaic surface in the spring of 1911, but apparently this mosaic is slowly disappearing by reason of the pitch rising in warm weather and sealing up the surface.

The remarks made in connection with Section 17 will apply to Section 18, except that the action and results referred to were earlier.

Section 19 bled badly in warm weather, and was extremely slippery

PARK HEIGHTS AVENUE, BALTIMORE, MD.

Mr. Crosby.

Square yards.	Dates of use.	Construction of Macadam per Square Yard. Gallons used.	Cost of resurfacing.	Cost of pitching, including chipping.	Total first cost.	Maintenance per Square Yard of Macadam. 1910.	Total.	Earthwork.	Painting edges.	Oiling and chipping.	Net actual repairs.
11 226.11	July, Aug., '09	2.39	$0.337	$0.327	$0.664	$0.088	$0.9011	......	......	......	$0.9011†
1 936.00	Aug., Sept., '09.	3.97	0.339	0.434	0.773	0.080	0.0455	$0.0455	......	......	0.0000
1 242.66	Sept., '09.	3.22	0.336	0.418	0.754	0.187	0.9947	......	......	......	0.9947†
1 173.33	Sept., '09.	3.67	0.337	0.449	0.786	0.081	0.0093	0.0093	......	......	0.0000
1 712.00	Sept., '09.	3.12	0.339	0.344	0.683	0.081	0.0506	0.0049	......	$0.0364	0.0093
1 826.66	Sept., Oct., '09.	4.19	0.333	0.606	0.939	0.079	0.0587	6.0024	......	......	0.0563
1 909.33	Oct., '09.	5.40	0.337	0.618	0.955	0.082	0.0268	0.0054	......	0.0180	0.0034
1 816.00	Oct., '09.	4.41	0.336	0.605	0.941	0.080	0.0195	0.0097	......	......	0.0098
1 170.66	Nov., '09.	4.46	0.340	0.454	0.894	0.091	0.0236	0.0187	......	......	0.0049
3 232.00	May, June, '10.	1.43	0.397	0.242	0.639	......	0.0177	0.0097	......	......	0.0080
5 981.33	June, '10.	1.25	0.397	0.264	0.661	......	0.0070	0.0028	......	......	0.0042
1 261.33	June, '10.	1.65	0.397	0.262	0.659	......	0.0353	0.0055	......	......	0.0298
4 685.33	June, July, '10.	1.70	0.397	0.327	0.724	......	0.0378	0.0029	......	......	0.0349
4 202.66	July, Aug., '10.	1.45	0.397	0.325	0.722	......	0.0158	0.0057	$0.0101	......	0.0000
3 184.00	Aug., Sept., '10.	1.69	0.397	0.292	0.689	......	0.0166	0.0048	0.0013	......	0.0105
5 808.00	Oct., '10.	0.61	0.397	0.084	0.481	......	0.0372	0.0037	......	......	0.0335
1 100.00	Sept., Oct., '10.	0.94	0.397	0.140	0.537	......	0.1050	0.0070	......	......	0.0980
3 568.00	Oct., '10.	1.42	0.397	0.325	0.722	......	0.0297	0.0136	0.0161	......	0.0000
1 893.33	Oct., '10.	1.70	0.397	0.287	0.684	......	0.0434	0.0091	0.0281	0.0062	0.0000
127.55	Nov., '09.	3.92	0.241	0.174	0.415	0.087	0.2929	0.0266	......	0.0484	0.2179‡
1 294.22	Oct., '09.	3.86	0.230	0.408	0.638	0.088	0.1528	0.0683	......	0.0332	0.0513
693.77	Oct., '09.	4.70	0.228	0.551	0.779	0.088	0.1991	0.0186	......	......	0.1805
976.88	Oct., '09.	2.95	0.229	0.405	0.634	0.082	0.2242	0.0391	......	......	0.1851
844.66	Oct., '09.	4.71	0.224	0.691	0.915	0.082	0.0900	0.0144	......	0.0293	0.0463
2 246.22	Sept., '09.	2.23	0.229	0.257	0.486	0.081	0.0669	0.0114	......	0.0041	0.0514
1 532.22	Sept., '09.	2.73	0.228	0.353	0.581	0.057	0.0854	0.0091	......	......	0.0763
3 758.22	Aug., '09.	2.98	0.216	0.303	0.519	0.057	0.0324	0.0038	......	......	0.0286
1 011.77	Aug., '09.	1.95	0.216	0.227	0.436	0.057	0.0147	0.0088	0.0055	......	0.0004
740.00	Oct., Nov., '10.	1.46	0.369	0.414	0.783	......	0.0232	0.0125	0.0079	......	0.0028
13 369.33	Oct., Nov., '10.	0.56	0.337	0.103	0.440	......	0.0447	0.0139	......	......	0.0308

† Reconstructed, 1911. ‡ Material gratis.

in cold weather, which accounts for the large expense for putting in extra stone (under "Repairs") and for "Oiling and Chipping."

Section 20 bled somewhat, requiring additional stone, and was slippery enough to require oiling and chipping.

Sections 21 and 22 required the addition and rolling in of considerable stone in 1911. The surface rutted and began to move laterally, but the application of more stone at different times seemed to remedy these defects.

Section 23 was so slippery in cold weather as to require oiling and chipping.

Mr. Crosby.

TABLE 5.—TRAFFIC CENSUS, PARK HEIGHTS AVENUE, 1910–1911, DURING THE PERIOD, 7 A. M. TO 7 P. M.

Class of traffic.	SECTION 1.						SECTION 2.								SECTIONS 17 TO 21, INCLUSIVE.						SECTIONS 28 AND 29.			
	Nov. 14, 1910.		Nov. 17, 1910.		Oct. 19, 1911.		Nov. 15, 1910.		Nov. 18, 1910.		Oct. 14, 1911.		Oct. 21, 1911.		Nov. 22, 1910.		Nov. 26, 1910.		Oct. 25, 1911.		Jan. 14, 1911.		Jan. 16, 1911.	
	No. movements.	Maryland units.	No. movements.	Maryland units.	No. movements.	Maryland units.	No. movements.	Maryland units.	No. movements.	Maryland units.	No. movements.	Maryland units.	No. movements.	Maryland units.	No. movements.	Maryland units.	No. movements.	Maryland units.	No. movements.	Maryland units.	No. movements.	Maryland units.	No. movements.	Maryland units.
One-horse vehicles.	225	450	265	530	263	526	202	404	188	376	211	422	188	376	26	52	24	48	26	52	10	20	14	28
Two- " "	109	436	99	396	122	488	107	428	94	376	94	376	74	296	13	52	8	32	7	28	....	....	1	4
Three- " "	10	60	9	54	......	......	3	18	3	18	....	......	....	......	....	......	....	......	1	6	...	....	....	....
Four- " "	16	128	11	88	2	16	23	184	14	112	2	16	1	8	....	......	1	8	1	8	4	32	5	40
Six- " "	2	24	....	......	1	12	....	......	2	24	....	......	....	......	3	36	2	24	....	......	14	168	11	132
Totals........	362	1 098	38	1 068	388	1 042	335	1 034	301	906	307	814	263	680	42	140	35	112	35	94	28	220	31	204
MOTOR TRAFFIC:																								
Motor cycles.......	8	16	8	16	30	60	4	8	5	10	40	80	....	......	2	4	5	10	9	18	....	....	....	....
Runabouts..........	24	240		270	57	570	15	150	14	140	55	550	41	440	5	50	2	20	27	270	....	....	....	....
Four- or five-seat cars...............	126	2 520	155	3 100	395	7 900	132	2 640	150	3 000	404	8 080	327	6 540	33	660	45	900	57	1 140	....	....	....	....
Seven-seat cars....	58	2 320	74	2 960	159	6 360	46	1 840	63	2 520	132	5 280	159	6 360	14	560	23	920	80	3 200	2	80	....	....
Omnibuses or drays	2	40	5	100	26	520	7	140	7	140	20	400	32	640	3	60	2	40	4	80	1	20	....	....
Totals........	580	6 234	653	7 514	1 055	16 452	539	5 812	540	6 716	958	15 204	825	14 660	99	1 474	112	2 002	212	4 802	31	320	31	204

Mr. Crosby.

Section 28, like Section 15, required some patching of small holes, but not to as great an extent. Between May 15th and July 1st, 1911, more than 1 000 tons of stone were hauled over this section (and Sections 26 and 27) for the construction of a branch road, in addition to the regular traffic shown by the traffic census, Table 5.

Section 29 was treated twice during 1911 by further applications of solutions the same as the original material. These solutions were made of 2 parts Glutrin and 3 parts water, and the mixture was applied to the center of the road each time, at the rate of 0.3 gal. per sq. yd.

All sections, except Section 2, are now in first-class condition and, as far as the macadam is concerned, need no repairs.

In Table 4, the column, "Earthwork," shows expense (per square yard of macadam) on shoulders, gutters, etc. The column, "Painting Edges," shows expense per square yard of macadam for this item in those cases where (because the pitch failed to flow out sufficiently to the sides) the edges of the macadam began to deteriorate under traffic. The column, "Oiling and Chipping," shows expense per square yard of macadam for remedying slipperiness by a light coat of cold, thin pitch, followed by a coat of stone chips. The column, "Net Actual Repairs," shows the expense per square yard of macadam for actually repairing the defects.

Mr. Tillson.

G. W. Tillson, M. Am. Soc. C. E.—The speaker hardly agrees with the statement that there is nothing in common in the work of city, State, and town engineers. The papers and discussions presented here are of great value, as they are based on the results of practical experience. If we know why a certain road was a success, or why it was a failure, then certain principles can be deduced which may be applied to the construction of different kinds of roads, and correct results can be obtained.

The Borough of Brooklyn, New York City, has a certain amount of imported rock-asphalt pavement. Three or four streets, all in the residential section of the city, were laid with this material in 1895. They cost more than those of Trinidad, California, or Bermudez asphalt, yet the speaker feels that they are better for heavy traffic. As a whole, these pavements have cost less for repairs than those of ordinary asphalt, although they have been down 16 years.

It has been necessary, in making repairs, to use the ordinary sheet-asphalt mixtures, but the speaker believes these repairs have cost less than if they had been made with rock asphalt similar to the original; at least, the unit cost has been less, and the repairs have proven satisfactory.

Mr. Gage.

R. B. Gage, Esq.*—It is not the speaker's intention to give the origin, theoretical composition, or geographic distribution of that

*Chemist, New Jersey State Road Dept., Trenton, N. J.

Mr. Gage. large class of organic compounds known under the general term of bitumens, but to consider the use of various compounds in road construction to-day, and state a few of the reasons why they have not always fulfilled the functions expected of them in a satisfactory manner.

There are few industries to-day of such magnitude as those of asphalt or coal-tar, which depend so much on chemical reactions and principles, in which the chemistry of the materials used is not better known and does not more definitely control the product than is the case with these two substances. The nature of the ingredients composing them makes a separation of the beneficial from the harmful almost impossible. It is only by making a series of comparative tests, some of which are more physical than chemical, that it is possible to define these mixtures, and even then the operator should have an extended experience, both in the laboratory and in actual construction work, in order to interpret the meaning of the results secured by these tests.

It is this lack of chemical precision or positiveness that makes it so easy, and sometimes convenient, to charge the failure of a bituminous pavement or road to the quality of the bituminous material used. The chemist is expected to make his tests with the utmost care and precision, and is often very severely censured if his results differ slightly from those of another operator. It appears to be taken for granted that a chemist should not make mistakes, that he should produce almost any kind of material that is needed, of an ideal quality, whether such a thing is possible or not, and that he should be censured for all his failures and indirectly for most of the failures of bituminous materials in general. It appears to be human nature to shift the responsibility for a failure to other shoulders than our own, and in this respect there is nothing unnatural about the average contractor or engineer. If a failure occurs, the contractor may not care to admit he has not lived up to the requirements of the specifications, or the engineer may not care to admit that such specifications have not been properly drawn, and therefore the bituminous material is blamed at once as the cause of the failure. However, before passing judgment on the cause of a failure, the chemist's methods will be followed a few steps farther, and the situation analyzed a little more in detail.

In the specifications, great care is generally taken to define the bituminous material and its application. As an extra safeguard, firms supplying such material are often required to give its name, origin, and method of preparation. Apparently, every precaution is taken to secure first-class material, to guard against errors in construction, and to make doubly sure that a first-class pavement will be built. The chemist, from now on, is supposed to check up the samples of bituminous cements as they are sent to him by the inspector or engineer, for it is very important that this particular ingredient of the pavement be kept up to the requirements as specified. What about the other ingredients of the pavement and conditions which are as important as the quality of the bituminous material, if a first-class pavement is to be produced, and over which the chemist has no control? After the contract has been awarded, the contractor discovers that he can secure some "just as good" material at a lower price than that specified, and he is allowed to use it. Often the bituminous material is improved by the use of some "just as good" flux, or even the bituminous material itself is replaced by a better grade until it is time to take another sample; or, perhaps the contractor cannot afford to wait longer for warm or clear weather, even if his mixtures do get stiff before they can be properly rolled into place. The "run of the crusher" is substituted for the graded sizes of stone specified, that is, half the stone will be coarser than desired and the other half finer. The voids in the former sized stone may be twice as great as in the latter, yet the same quantity of bituminous material per square yard or ton is used. The quantity of dust that screenings shall contain is generally limited in the specifications, but this limit is often forgotten in wet weather, and screenings with an excessive dust content are used. Nevertheless, the bitumen is supposed to digest this excessive quantity of dust when the weather conditions are not favorable for producing clean stone. Mr. Gage.

These are a few of the details, and it is just as important and necessary that they be kept up to a definite standard, if a good pavement is to be produced, as it is to have a high-grade bituminous material. Neglect in taking proper precautions and maintaining required conditions, which are absolutely necessary to produce a good pavement, has caused more failures than the use of inferior grades of bituminous materials. It is possible with careful manipulation during construction, and good road metal properly graded, to build a fairly good pavement with even an inferior grade of bituminous material; on the other hand, the best of bituminous materials will prove a failure if proper methods of construction are not maintained, and if the mineral aggregate is not kept of the right composition and quality. This failure to keep the methods of construction and the other ingredients of the pavement up to a definite standard of quality is no doubt the chief cause of the pessimistic view many engineers now have regarding the value of chemical tests in determining the quality of a bituminous material. A few years ago many appeared to have the opinion that, no matter how the bituminous material was incorporated into the road, it would perform the functions required of it in a satisfactory manner. A few failures soon proved the absurdity of such views or opinions, and made many very skeptical regarding bituminous roads in general.

Mr. Gage. Not long ago the speaker listened to an address in which the opinion was expressed that chemical tests were of no value whatever in determining the merits or the qualities of either bituminous or Portland cements, and that the latter never was of much value until its properties were defined by physical tests; also, that the same method of testing should be applied to bituminous pavements, that is, it should be confined to the finished pavement. It is very evident that, in this case, the speaker did not know that all the material used in the manufacture of Portland cement is analyzed first and the mixture is kept as near a definite chemical composition as possible; also, that Portland cement, after it has once taken a set, is practically immune to the action of surface water, while bituminous cements are soon ruined if not protected from these agencies. Almost any bituminous material possesses sufficient strength, when of the proper consistency, to hold the average mineral aggregate together under the worst conditions, provided the life of the material can be maintained. Comparative tests of bituminous and Portland cement mixtures, made to-day, might show high value for the former and low for the latter, but if the same tests are repeated after both materials have been in use for a couple of years, the reverse might be true. During this time the surface waters may have totally disintegrated the bituminous cement, but may not have affected the Portland cement in the least. Certain tests may define the quality of Portland cement, yet, if used to determine the life of a bituminous pavement, they would be more or less unserviceable, unless accompanied by other tests which would show the ability of these pavements to resist the action of surface waters.

There is little doubt that 75% of the failures of bituminous pavements during the last few years, for which the bituminous material has been given the full blame, can be traced to the use of inferior grades of road metal, faulty methods of construction, or combinations of these two causes. In making this statement it is not claimed that all bituminous materials, even when of the proper consistency, are of equal value in road building. Some failures are caused by the use of inferior grades of bituminous material, but these are often used simply because they are cheap, and not because they have been recommended by the engineer or chemist in charge.

Naturally, such failures tend to condemn the use of bituminous materials for similar purposes, and make the securing of contracts for a better and higher priced material often a very difficult task. These failures are frequently explained so nicely, and with such plausible excuses, that they are often repeated in the following year, if the bituminous material used can be shown beyond a doubt to be the best on the market and a great bargain at the price. It is only after the real causes have been correctly interpreted and determined

Mr. Gage.

that it is possible to apply the remedies which will prevent a repetition of such mistakes. It may be of financial benefit to certain parties at given times to misrepresent the real cause of these failures, yet this only makes those who are responsible for this particular type of pavement all the more pessimistic, and many times completely kills the chances of similar pavements being again specified. If the chemists or their tests are responsible, the speaker is sure they will gladly shoulder the blame. If the fault lies with the bituminous material, which happens to be of the bargain variety, it is false economy to use it at any price. If the road metal is not of the proper quality or correctly graded, if the methods of construction or preparation of the pavement are faulty, or if the contractor or his men are inexperienced, do not try to shift the blame to the bituminous material or the chemical tests, even if it is very convenient to do so, but place it where it belongs, for only by proceeding in this manner will the real cause of these failures ever be eliminated, and the bituminous pavement be placed in its proper position.

Mr. Dunham.

FREDERICK DUNHAM, ESQ.—The speaker has had charge of the construction of bituminous pavements by the penetration method on the Hudson County Boulevard for several years.

In 1907 about 40 000 sq. yd. of bituminous road and an equal area of water-bound macadam road were built in practically the same section for the purpose of ascertaining the difference in the cost, and comparing the life and wearing qualities of the two.

The surface course was composed of $1\frac{1}{2}$-in. stone filled with screenings, which were broomed off before the application of the Tarvia, about $\frac{2}{3}$ gal. being used for the first application. This coat was covered with a thin layer of $\frac{3}{4}$-in. stone and on this was spread the second coat of Tarvia, about $\frac{1}{3}$ gal. per sq. yd. This coat was covered with screenings containing about 50% of dust. The water-bound macadam road was of the ordinary type, with $1\frac{1}{2}$-in. stone and screenings in the surface. This road was finished in July, 1907, and was in excellent condition up to about June, 1908, when the heat of the sun caused the surface to wave and roll into bunches. There were no signs of disintegration, however, and the road held together except for the waving, until the spring of 1911, when it was necessary to reconstruct it. When the material was taken up, it showed that, though only 1 gal. per sq. yd. had been used, the Tarvia had penetrated to a depth of about $1\frac{1}{2}$ to 2 in., and still had life in it. The macadam, which was laid at the same time, lasted about 8 months; at the end of that time it had all raveled and gone to pieces. The difference in the cost of the two roads was 10 cents per sq. yd.

In 1908, about 100 000 sq. yd. of bituminous macadam were laid in practically the same manner as in the previous year, except that the quantity of Tarvia in the first application was 1 gal., and in the

Mr. Dunham. second application ½ gal. per sq. yd. This road lasted about a year At the expiration of that time, though the surface had not waved much, it had commenced to ravel. Apparently, there had not been enough binder in the second coat. The road disintegrated in places, and had to be repaired at an expense of from 10 to 12 cents per sq. yd.

In 1909 about 100 000 sq. yd. of pavement were laid by exactly the same method as adopted in 1908, except that a better grade of Tarvia was specified. This road was not much more satisfactory than that laid in 1908. The surface had a tendency to creep, and it disintegrated in places, apparently due to unequal distribution of the binder, or because at these points a sufficient quantity of binder had not been used. The bituminous material used in 1908 and 1909 did not seem to hold its life like that used in 1907.

In 1910 about 50 000 sq. yd. of pavement were laid with Tarvia, and about 50 000 sq. yd. with Standard Asphalt Company's Binder A, using 1½ gal. per sq. yd., as before. The work built with Tarvia was in good condition for about one year, possibly 18 months; the road built with Standard Asphalt Binder A is in good condition to-day, and has not needed any repairs. The binder has come to the surface, so that there is no stone visible, and the road looks almost like a sheet-asphalt pavement.

Since 1910, the method of construction has been changed. The first course is composed of 1½-in. stone, and is firmly compacted, the voids being filled tight, as in an ordinary water-bound macadam. On this surface is placed a layer of 1½-in. stone which will be 2 in. thick after rolling. This layer, however, is not filled, as was formerly done. The bituminous material is poured on this course and then covered with ¾-in. stone in a sufficient quantity to fill the voids. Another application of bituminous material is made, and the resulting surface is covered with screenings. Standard Asphalt Binder B was used: 1½ gal. for the first application and ½ gal. for the second. This work lasted well for about 6 or 8 months, but after that the ¾-in. stone became exposed, and the road looked as if it was about to ravel. A seal coat, using from ⅝ to ⅞ gal. per sq. yd., of the Standard Binder A, was applied and covered with about 1 in. of Cow Bay sand. This treatment saved the road, and it is in good condition to-day. On about 45 000 sq. yd. of road 3 gal. of bituminous material were used per sq. yd.; 2 gal. for the first application and 1 gal. for the second. This second coat, however, was covered with dustless screenings or with pea-sized stone in sufficient quantities to take up the asphalt binder. This road looks like a sheet-asphalt pavement, and as if it is going to give very good results.

The best piece of road constructed by the penetration method, up to 1910, was built during the winter. Construction was started on

November 1st, but was stopped about the beginning of January on account of snow. As soon as the snow disappeared and the road had dried up, work was continued and was finished about March 1st. The bituminous material used was Tarvia X. Naturally, the tar did not penetrate much, and in the spring there was some bleeding. The spots were covered with screenings and rolled, and the road is in as good condition to-day as when it was finished. It does not show the slightest sign of disintegration, but looks like a sheet-asphalt pavement. About December 26th, 1911, 25 000 sq. yd. of road, built by the penetration method, using 3 gal. of binder per sq. yd., were finished. All the work was done in November and December, and, from his past experience, the speaker expects it to be one of the best pieces of road that he has built. Weather conditions, therefore, may not have as much effect on the construction as supposed. The speaker believes that, when bituminous material is applied in extremely hot weather, it is likely to penetrate too deep, and that the binder will do more good up near the wearing surface than below. It has been a serious question with him as to whether roads built by the penetration method have many great advantages over ordinary water-bound macadam roads with a surface treatment, though it may be that the cost of maintenance of a road built by the penetration method will be a little less than for resurfacing ordinary water-bound macadam roads. Mr. Dunham.

The penetration method is only a substitute for a bituminous pavement built by the mixing method, and was first used with the idea of economy. Ultimately, the mixing method will have to be used if pavements are to be built to last for any great length of time. If pavements built by the mixing method can be constructed at practically the same cost as those built by the penetration method, as has been stated, there is no doubt that the former method will be by far the more economical.

H. C. POORE, JUN. AM. SOC. C. E. (by letter).—It has been stated that the voids in broken stone, spread and rolled ready for the bituminous application in the customary penetration method, vary from 20 to 40 per cent. The writer has also found this to be true, although he has made no determination of the actual voids. In one instance, on two adjoining square yards of stone surface, 2½ in. thick, 1.25 gal. per sq. yd. on one, and 3.7 gal. per sq. yd. on the other, were flushed over the surface, and gave the same results in appearance. This showed the voids to be 60% greater in one than in the other. Mr. Poore.

The writer has had considerable difficulty with uniform bituminous distribution on this account, as all parts of the stone surface certainly do not require the same quantity of bituminous material per square yard. As long as stone is spread by hand, there will be an unequal percentage of voids, and hence it is true that a skillful operator, spreading the bituminous material, either by single-spray nozzle or

Mr. Poore. pouring pot, can vary the quantity per square yard, which will result in a uniform surface. This operator should be a high-priced workman capable of understanding the result desired and, by alertness of the eye, obtaining it. The spreading device must necessarily be one which will allow the surface to be coated to be easily seen, and should deliver the material uniformly, allowing the variations to be made entirely by the operator. It is the writer's belief that many failures in penetration work are due to this inequality of stone voids not being considered.

Mr. Oxholm. THEODOR S. OXHOLM, M. AM. SOC. C. E. (by letter).—For the past five years bituminous concrete pavements, by the mixing method, and, for the past two years, bituminous macadam pavements, by the penetration method, have been laid to a considerable extent in the Borough of Richmond, New York City. The experience gained by the writer in these works, both from observation and the actual handling of many paving contracts, as well as from the inspection of various kinds of bituminous pavements in different parts of the country, has forced him to the conclusion that the mixing method is far superior to the penetration method. It is also believed that the difference in cost between these two methods is now so slight as to be negligible when considering which is the better type to use. In the mixing method the voids are much less in volume than in the other, and this, of course, means longer life.

Six years ago an original specification was made by the writer for the mixing method. Several contracts were advertised, but injunction proceedings in the State Courts prevented the opening of bids for about a year. The City was then permitted by the Appellate Division to open bids and award the contracts. The streets have now been in service for about five years. No repairs have been made, and none are necessary, though one of these streets has a single-track car line through its center, and the travel is moderately heavy and includes all classes of vehicles. At present the City is enjoined from using this specification, the injunction having been obtained by the Warren Brothers' Company, who claim violation of its patents.

Last year 46 000 sq. yd. of bituminous concrete pavement were laid in the Borough of Richmond, according to what is known as the "Topeka" specification, which is as follows:

"Sec. 59. Bitumen from 7 to 11 per cent.

"Mineral	aggregate	passing	200-mesh	screen,	from	5 to	11	per cent.
"	"	"	40- "	"	"	18 "	30	" "
"	"	"	10- "	"	"	25 "	55	" "
"	"	"	4- "	"	"	8 "	22	" "
"	"	"	2- "	"	less than		10	" "

"Sieves to be used in the order named."

This pavement has been laid 2 in. thick on both a concrete and an old macadam base, which was in good condition. The United

States Supreme Court has decided that this pavement is not an infringement on the Warren Brothers' patent. It was found that this specification permitted the use of about 75% of the mineral aggregate, stone, and 25% sand, or 25% stone and 75% sand, and many other combinations between these two percentages. The new specifications covering this point read as follows: Mr. Oxholm.

"The mineral aggregate shall be composed of trap rock screenings and sand, which shall be used in proportions to be determined by the Engineer from samples of materials to be furnished by the Contractor, and approximating 3 of screenings and 1 of sand, within the requirements of Section 59, and shall be heated in mechanical revolving dryers, at a temperature not exceeding 325° Fahrenheit; after which the asphaltic cement at the proper temperature and in the proper proportion shall be added, and the entire mixture then placed in revolving mixers and thoroughly agitated until all particles of the mineral aggregate are thoroughly and completely coated with the hot asphaltic cement. The mixing shall be continued until the combination is a uniform bituminous concrete.

"After this mixture is delivered where same is to be laid, it shall be evenly spread with hot iron rakes to such a depth that it will have a thickness of two (2) inches after having been thoroughly rolled with a steam roller weighing not less than 12 tons.

"After the pavement has been rolled with a steam roller the surface of same shall be swept and covered with clean, dry sand in order to completely fill any surface voids which may exist."

It is desirable that the pavement be constructed with as much stone as possible, provided a small amount of voids exists when combined with the sand. There is very little trouble from slipperiness, and moderately steep crowns may be used, forcing storm-water to seek the gutters quickly.

Adjoining the curbstones no bituminous pavement should be laid, but stone, brick, or other kind of pavement not susceptible to the action of water or other matter, should be substituted. In the writer's early experience with pavement of this class, it was laid 3 in. thick. This was found to be objectionable, because the pavement would creep in warm weather in many places. The thickness now generally accepted as proper is 2 in. The asphalt specifications for this work should permit the use of Trinidad Lake asphalt, as well as Bermudez and properly refined asphaltic oils. All the mixing should be done in a semi-portable mixer, the stone being heated before the application of the asphaltic cement.

The resulting evenness of mixture is so far ahead of anything which can be obtained by the penetration method, and the fact that the latest form of machinery has brought the cost down to from 3 to 6 cents for mixing, and that these machines only cost about $2 000, as stated by Professor Blanchard, that there can now be no legitimate reason for the further building of bituminous roads by the penetration method.

Mr. Howard.

J. W. Howard, Esq.—In Mr. White's discussion is this sentence:

"The additional cost of surfacing by the mixing method over first-class construction by the penetration method is so slight, and the advantages of uniform results and longer service attained are so considerable, that every municipality should look carefully into it before deciding on the inferior method."

It is the speaker's experience, and without doubt the experience of all, that in engineering construction of all kinds, the cost of maintenance must be borne in mind.

A number of States have borrowed money on 20- to 50-year bonds for the construction of roads and pavements, but these will have been worn out many years before the bonds are paid. This is going on all over the country, and has created indebtedness which must be met by a future generation which will not have used the roads or pavements. Low maintenance cost, and not low first cost, is true economy; therefore, the cost of upkeep and constant maintenance must be included when money is obtained from loans for construction purposes. A loan should not be made for a longer time than the life of the structure for which it is obtained.

The speaker believes that engineers should do all in their power to build durable roads and pavements, and that, with bituminous or asphaltic surfaces, this will be more nearly accomplished by using mixing methods, for which the materials can be standardized. There are a number of tests, both physical and chemical, which are of great value. Interpretation of the results obtained from such tests as to the wearing qualities or adaptability of any material must be based on experience; for instance, if an asphalt having a certain softness, ductility, adhesion, etc., has been in successful use for twenty years, such tests may be regarded as sufficient for that material.

In several States, during the past year, the furor for constructing roads has been such as to cause the waste of enormous sums of money. After four years from one-fourth to one-sixth of the first cost of construction will be required for the annual maintenance of some of these roads, whereas if a small portion of this money had been put into better original construction, the later annual expense would be decidedly decreased, and eventually, appropriations would be for maintenance only. In Paris, France, every dollar appropriated for pavements, during the last forty years, has been for maintenance only.

Some time ago, Broadway, New York City, was being repaved with granite blocks, and an engineer from Berlin told the speaker that he found it was being wrongly charged to repavement fund. It should have been charged to maintenance or repairs, for it is just as much repair work as the replacement of old rails on any railroad.

The Wadsworth macadam is simply crushed bituminous sandstone incorporated in an ordinary broken-stone road. The first successful

Mr. Howard.

asphalt (bituminous limestone) pavement was laid in Lyons, France, about 1838, with material from the Seyssel Mines, on the Rhône. The first fully successful street pavement built with this material was laid in 1854, on Bergère Street, Paris. Many more pavements were laid in Paris at about the same time, and these have been given constant and proper attention and are in excellent condition to-day. The statistics and records of these pavements are contained in the *Annales des Ponts et Chaussées.*

Except in Indian Territory (now Oklahoma) and Utah, the bituminous limestone of Europe has not yet been found in America. Inasmuch as this entire material must be transported to the place where it is to be used, it is absolutely out of the question to consider it for general use in pavements far distant from the quarries. The bituminous limestone of Oklahoma should not be confused with the bituminous sandstone of Oklahoma, Kentucky, California, etc., which is a very different material. In Oklahoma, however, the political and economical crime was committed of drawing specifications calling for asphalts from far distant sources, such as Trinidad, and excluding Oklahoma and other good American asphalt paving materials. Oklahoma cities have borrowed $60 000 000 for the construction of pavements, and, to-day, many of these are cracking and going to pieces. None was built with the Oklahoma bituminous limestone.

This bituminous limestone is practically a marble impregnated with bitumen. The only objection to it is that it makes a very slippery pavement. As used in all European cities, it is crushed, ground, or disintegrated by machinery, so that it will pass through a 20-mesh screen, then heated to about 120° Fahr., and rammed, not rolled with a steam roller. The Germans call it "Stampf Asphalt" and the French, "Asphalte Comprimé." The American sheet-asphalt pavement is an artificial bituminous sandstone, consisting of about 80% of sand, 10% of limestone powder, and 10% of asphalt-cement.

Such artificial asphalt mixtures have been improved during the past twenty years, so that now it is possible to lay a pavement which will last for a long period with little repairs. A good asphalt pavement on a concrete foundation is one which, with close attention, can be kept in good condition for any period of time.

Referring to foundations, there are many miles of asphalt pavements in Buffalo, N. Y., and elsewhere, the concrete foundations of which are very pervious to water. The water distributes itself under the wearing surface, and suddenly 2 or 3 miles of pavement will crack in pieces. When these pieces are examined, the decayed matter can be easily seen. This shows the necessity of constructing foundations which will keep dry, for they should be under-drained so that water can pass readily from them, or they should be so dense that it will not come up through them.

Mr. Howard. The binder course was introduced into the construction of asphalt pavements in order to keep the surface layer from shoving or slipping on the foundation. By reducing the bitumen in the original pavement mixture to about 5% in the binder, a considerable saving was effected; but a loose and open binder was the result, so that water comes down through incipient cracks in the wearing surface, and, in New York City, decays the surface layer in many streets.

In laying a binder course, whether on a country road or a city pavement, it should be close, dense, and impervious to water. Refined asphalts, asphalt-cements, asphalt road binders, etc., should not be used when they are softened, soluble, or affected by water. The "water-weakness test," which should be in all specifications for bituminous materials, would prevent the use of a few impure refined asphalts, which are often extensively used, but are seriously injured by water. In this test a piece of glass is painted with hot refined asphalt, or asphalt-cement, and immersed in water for about 10 days. If affected by water, the coating will soften, turn gray and then brownish, and lose its adhesive quality, so essential in bituminous and asphalt pavement mixtures.

Mr. Driscoll. MICHAEL DRISCOLL, ESQ.*—In 1908, owing to the construction of a Metropolitan sewer some 7 ft. in diameter, it was necessary to resurface Chestnut Street, in Brookline. The subgrade was of a gravelly nature. The road was constructed originally of ordinary pit gravel, and had been maintained for 25 or 30 years by patching and resurfacing with broken stone whenever necessary. It carries a large volume of traffic, most of which is heavy automobiles moving at a rapid rate.

The road was shaped and rolled to an even grade 2 in. below the finished grade. The wearing surface was composed of trap rock, 3 in. thick, which was rolled down to 2 in., three methods being used in its application.

In the first experiment, a layer of No. 1 stone, $1\frac{1}{4}$ to $2\frac{1}{2}$ in. in longest dimensions, was laid for a distance of about 650 ft., and was treated with the American Tar Company's tarite asphalt by the penetration method, using about $1\frac{3}{4}$ gal. per sq. yd. In the second experiment, for a distance of about 450 ft., No. 1 and No. 2 stones were mixed, the No. 2 stones varying from $\frac{3}{4}$ to $1\frac{1}{4}$ in. in longest dimensions. This piece was also treated by the penetration method, and with the same bituminous material. In the third experiment the remainder of the road, about 700 ft. long, was constructed by using No. 2 stone mixed with tarite asphalt before placing. The mixing was done on the road, and the stones were thoroughly coated with the bituminous material. The road was then thrown open to travel, and nothing was done to it for the remainder of the season.

* Superintendent of Streets, Brookline, Mass.

In the spring of 1909 the part built with No. 1 stone by the penetration method began to disintegrate in spots where the bituminous material was insufficient. These places were repaired, and a flush coat was then applied to the whole length of the road. Since that time not a cent has been spent for repairs. In passing over the road at the present time it is impossible to notice any difference between the three sections. Although some very good results have been obtained in Brookline by the penetration method, the speaker believes that the mixing method will produce a more durable surface. Mr. Driscoll.

HERBERT SPENCER, ASSOC. M. AM. SOC. C. E.—The sizes of the aggregates used in the different courses in the penetration method are of great importance. Unfortunately, however, there seems to be a great diversity of opinion in respect to the arrangement of the sizes of stone, and numerous specifications which have come before the speaker show an urgent need of standardizing this feature. Mr. Spencer

As illustrating this point, extracts are made from a number of specifications, as follows:

(*a*) "The top course stone, consisting of the run of the crusher, from screenings to and including 2-in. stone, shall be spread on the bottom course to such a depth that it shall have, when completed, the required thickness, after which 1¼ gal. of bituminous material shall be evenly spread over the surface. * * * After the bituminous material has all been applied, sufficient additional screenings shall be added to the surface to fill the voids and cover the road thinly. * * * The loose screenings shall then be swept off with hand brooms, after which ⅓ gal. of bituminous material to each square yard shall be evenly spread over the surface in the same manner, and immediately thereafter be covered with ⅜ in. of dry screenings."

(*b*) "As soon as the second course has been sufficiently compacted and the voids filled with the finer sizes, the surface shall be thoroughly cleaned of all surplus fine material by brooms or such other measures as may be necessary to secure the result. * * * On the warm, dry, and cleaned second course shall be spread a coat of pitch compound in such quantity as will thoroughly saturate the prepared surface. * * * After the pitch compound has been applied, the pitched surfacing will be lightly coated with clean screenings, sand, or stone chips."

(*c*) "Upon the telford foundation there shall be spread a binder course of stone, which shall pass a 2½-in. screen and pass over a ¼-in. screen. This course shall be thoroughly rolled, and sufficient screenings added to make a thoroughly compacted and smooth surface. * * * Upon this surface shall be evenly spread 1½ gal. of bituminous material to each square yard. * * * Immediately thereafter a course of 1½-in. run of the crusher stone shall be evenly spread on the surface. * * * A layer of the same bituminous material, ½ gal. to the square yard, shall be spread evenly over the surface, and the same covered with sufficient dry, dustless screenings to take up all excess bitumen."

Mr. Spencer.

(*d*) "The second course of 1½-in. stone shall be spread and rolled as directed. The voids shall then be filled with dustless screenings and ¾-in. stone until about 75% of them have been filled. * * * Bituminous cement shall then be spread in a uniform layer at the rate of .... gal. of bitumen per square yard. A layer of ¾-in. stone and dustless screenings shall be spread at once and the road rolled. A second application of bitumen shall then be made, covered at once with a coat of screenings and the road again rolled."

(*e*) "Upon the properly prepared bottom course there shall be evenly spread 1½-in. stone to a depth of 3 in. The course shall be dry-rolled, the surface being open or porous in order to allow the penetration of the hot binder. Directly after the application of the binder, clean, dry ¾-in. stone, free from dust, shall be spread over the surface in sufficient quantities to fill the surface voids completely. * * * A seal or flush coat of the hot binder shall be uniformly distributed over the whole surface. * * * Clean, dry ⅜-in. stone chips shall then be spread over this seal coat in just sufficient quantities to take up all excess binder, leaving a slight excess of stone chips to protect the surface while setting up."

In these specifications it is noted that *a*, *b*, *c*, and *d*, in one way or another, call for the filling or partial filling of the voids with finer particles of sand or screenings before applying the bitumen. On the contrary, *e* calls for the top course to be left "open or porous, in order to allow the penetration of the hot binder." It may also be stated that all these specifications call for a binder of about the same consistency, the difference in penetration or viscosity being but slight. It is readily seen, therefore, that great differences in the final appearance of the road will result from the variety of methods suggested, and, to the speaker, the point is one which seriously affects the success of a road under this method.

In the first four specifications, *a, b, c,* and *d,* it is evidently intended to produce as dense a mineral aggregate as possible before the application of the bituminous binder. It is further presumed that the screenings, both under and over the bitumen, will become incorporated with the latter after rolling. It is felt, however, that this is almost impossible to obtain in practice, as any appreciable quantity of stone screenings or sand in the 1½-in. stone will act as a blanket, and not allow the bituminous binder to penetrate properly. The consequence is that the bitumen, together with the final course of stone screenings, is left on the top of the road without any real bond to the underlying stone surface. When this condition takes place, there is danger of the traffic pushing the superimposed layer of screenings and bitumen, and causing the road to have a wavy appearance. The surface is also likely to develop weak spots, due to the uneven sizes of stone, as it is very difficult to maintain the proper assortment of sizes in either a dumped pile or by casting the stone.

Fig. 8.—Bituminous Macadam Road, Penetration Method, Standard Macadam Asphalt Binder, New York State Specifications. Huntington-Farmingdale Road, Long Island, Built 1909. Sample Taken from Road, November, 1911.

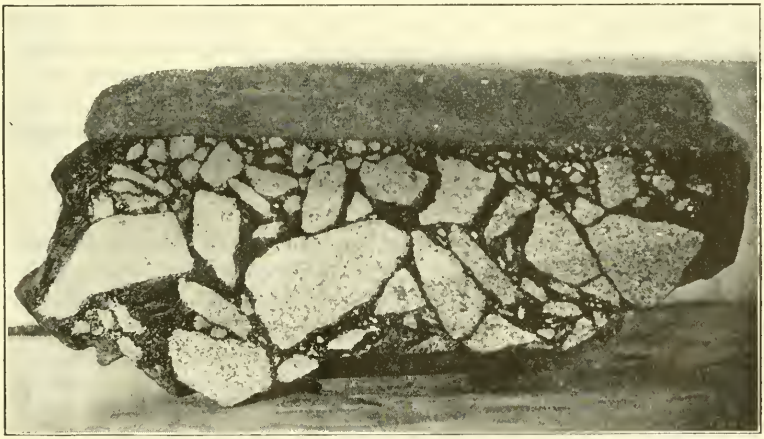


Fig. 9.—Bituminous Macadam Road, Penetration Method, Standard Macadam Asphalt Binder "B." Bentley Street, Tottenville, Staten Island, Built 1909. Sample Taken from Road, November, 1911.

The main factors in the penetration method of construction are: Mr. Spencer.

1.—Quality of stone.

2.—Proper sizing of stone for each successive course.

3.—Penetration or viscosity of bitumen used.

After comparing roads constructed under specifications similar to *a, b, c,* and *d* with those constructed according to *e,* the speaker is convinced that from both theoretical and practical standpoints, the latter is the proper method to use. This provides for an alternate layer of stone and bitumen, decreasing the size of the stone used for each course. The stone, when rolled into the interstices of the next lower course, will become, to a certain extent, coated with bitumen.

It may be argued that, if no filler is placed in the 1½-in. stone, the bituminous binder will run through this course and lose its efficiency. This may be true only with an exceedingly soft bitumen; one ranging from 100 to 180 penetration will have a tendency to become chilled and regain its normal consistency before penetrating to the bottom of the course. If any filler is needed, it can be supplied by sufficient rolling, as the sharp edges of the stone are readily fractured under heavy rolling and will settle to the bottom of the larger stones.

Under the mixing method, the question of the proper arrangement of sizes of stone is comparatively simple. By this method any desired combination of stone can be mixed mechanically and the correct quantity of bitumen can be added to the mass. The mixture will retain its homogeneous character, and can be raked and rolled to the depth and shape desired. By the penetration method, it is not possible to accomplish this as easily, and, therefore, dependence must be placed on an alternate layer of stone and bitumen to produce the densest mixture possible.

After observing many hundreds of miles of roads in various parts of the United States, the speaker is convinced that one of the secrets of success in the penetration method lies in the observance of this feature. The operation of constructing a penetration road looks very simple, but, like many other things, apparent ease leads to carelessness and poor results. Many miles of excellent roads have been constructed under the penetration method, and, where proper attention has been paid to details, the results are as favorable as those on many types of more expensive pavements. That type of construction is well adapted to long stretches of road between large centers of population; it can be made dustless, is easy to repair, is low in first cost, and has proven its ability to fill a long-felt want. No expensive plant is required, but there is need of the most careful inspection and engineering ability.

Figs. 8 and 9 show sections of roads constructed in 1909 under specifications similar to Type *e.* Fig. 8 is a sawed section from the

Mr. Spencer.

Huntington-Farmingdale Road, Long Island, New York State. The aggregate used was a mixture of trap rock and copper slag. Fig. 9 is a sawed section from a road in Tottenville, Staten Island, in which the aggregate was also a trap rock. These roads are still in excellent condition, showing no signs of wear, and are smooth and dense. The bitumen used was the same in all sections.

The speaker was connected with the construction of one of the experimental sections of the White Plains Road, Borough of the Bronx. This section was built by the mixing method, using a binder manufactured by the Standard Oil Company. At the present time the surface is exceedingly wavy, and there seems to be considerable doubt as to the cause of this condition. The bituminous material used had a penetration of 120, and the aggregate consisted of stone ranging in size from 1½ in. to ¾ in., mixed with screenings and sand.

The heated mixture was deposited from wheel-barrows in windrows across the road. The tops of the piles were then struck off with shovels and rakes, and the rolling was done with a 15-ton roller—exceedingly heavy for this work. Under these conditions, it seems natural that, when rolled, there would be a greater degree of compression under the center of the pile than under the part struck off, which, therefore, would cause a wavy appearance. The depressions occur at about the same intervals as those between the original piles of material. As the mixture was carefully proportioned and was sufficiently dense, the speaker believes that the waviness was caused entirely by faulty construction, rather than by the softness of the bitumen or the sizes of stone used.

Mr. Sharples.

P. P. Sharples, Esq.*—The relative economy of the penetration and mixing methods can only be determined by local conditions. In the immediate vicinity of Boston the conditions are very favorable for penetration work, and in many cases where it would be out of the question, on account of the expense, to put in work by the mixing method, the penetration work is justifiable and very much to be desired. Some points in particular which should always be considered are the situation of the road, how the materials brought upon it can be handled, how much of the road can be fenced off during construction, and how long a time can be taken to do the work.

The conditions which must be met by different engineers are quite variable, and one who travels around the country realizes this a great deal more than one whose experience is confined to one particular locality. The state engineer, the town engineer, and the city engineer all have to contend with different conditions, although in the border line their work may be similar. Therefore, it is not to be expected

* Chief Chemist, Barrett Manufacturing Company, Boston, Mass.

that a method which has been successful with one should necessarily be successful under an entirely different set of conditions. Each problem must be thoroughly studied to determine the effect produced by difference in conditions. Mr. Sharples.

The method of distributing the bituminous material is probably of more importance in the penetration method than it is in surface applications. It has been the speaker's good fortune to see almost all the apparatus used for distribution work in the United States and abroad, and he has found that a machine which is perfectly adapted for surface treatments does not seem to be suitable for penetration work. A machine which distributes the material in a sheet, or a sprayer which covers a width of 6 or 8 ft. in one passage, is not as well adapted for penetration work as for surface treatment. A machine which delivers the material in a spray that hits the stone in the road at a high velocity seems better for both surface treatment and penetration work. In penetration work the speaker prefers to use a sprayer which delivers the material through a single nozzle of considerable capacity at a comparatively low pressure. This kind of a machine requires a well-trained man to handle the nozzle. The training of a man is not difficult, but it is much easier to train a new man than one who has had experience with other machines. There are many good roads in the vicinity of Boston, built by the penetration method with a spraying machine of this type, which are as perfect as they can be made with that method.

A bituminous pavement requires more constant watchfulness than an ordinary macadam road, on account of the quick ravelling which takes place as soon as the surface is broken. Therefore, it is very important to repair defects at once and keep the surface intact. When the surface becomes bare, and before it actually breaks, a seal coat of bitumen must be applied, in order to renew the life of the bituminous surface for a further period.

The speaker does not advocate building roads by either the mixing or the penetration method without a seal coat. This is particularly true of penetration work, as the seal coat is required to remedy defects in the surface. If an excess of bituminous material is used, it will rise to the surface and act as a flush coat, so that no extra seal coat is needed; but if the quantity of bituminous material has been kept down to the minimum required to bind the stone together, a seal coat is necessary. Occasionally, however, a road seems to have a charmed life and stands up without the seal coat. A road in Rhode Island, on the State Highway to Narragansett Pier, constructed in 1908, is remarkable for the way it has endured. It was built with a refined tar, by the mixing method, without a seal coat. Although it is subjected to the traffic of from 300 to 500 motor vehicles and about 50 horse-drawn vehicles a day, it is in perfectly good condition at the present

Mr. Sharples. time and has never required any expenditure for maintenance. In its great preponderance of automobile traffic, and in the absence of heavy trucking and small traffic in the winter, the road is exceptional. Such remarkable results could hardly be expected where the conditions were less favorable to a bituminous pavement.

Mr. Collier. H. L. Collier, Esq.—The speaker wishes to take issue with a number of engineers who seem to think that the penetration method of building roads has been a failure. The fault has been due, not to the bituminous materials, but entirely to lack of rigid inspection. In most cases the contracts are let by the square yard, and the contractor is so anxious to make a big yardage each day that the distribution of the bituminous material is carelessly done, and, in many cases, too small a quantity is used. One Sunday last summer, in making an inspection of a State road being built near New York City, the speaker found the contractor's forces working. There was no inspector on the ground, and, on the first course, where the contractor should have been pouring $1\frac{1}{2}$ gal. of asphalt, he was really only pouring about $\frac{1}{2}$ gal. This is one instance where the road will go to pieces, and the engineers will blame the penetration method for the failure. The best solution of the problem, if the work is to be done by contract, is to make contracts for the stone at a price per cubic yard in place, and for the bituminous material at a price per gallon in place. Any company manufacturing bituminous material would be glad to furnish duplicate bills to show that the contractor was buying the proper quantity of material. For the construction of roads by the penetration method, the speaker recommends the following:

On the base, prepared in the usual way, place $3\frac{1}{2}$ in., loose measurement, of what is commercially known as $1\frac{1}{2}$-in. stone, that is, stone which will pass through a $2\frac{1}{4}$-in. ring and will be retained by a $1\frac{1}{4}$-in. ring. Roll this lightly, and if, after this rolling, any depressions should appear, add more stone of the same size at such places. Over this, spread $1\frac{5}{8}$ gal. of the binder; then spread enough $\frac{3}{4}$-in. stone, that is, stone which will pass through a $1\frac{1}{4}$-in. ring and be retained by a $\frac{3}{4}$-in. ring, to fill the voids of the $1\frac{1}{2}$-in. stone. At this point most of the rolling should be done. After rolling, sweep away the loose material and pour $\frac{5}{8}$ gal. of binder on the road; over this. spread stone chips and roll thoroughly.

Hand-pouring pots, if properly manipulated, will give excellent results. The speaker is very much opposed to swinging the pots. because in that case it is almost impossible to get an even distribution, the quantity poured at the ends of the swings being almost double that at the middle. By walking and pouring first in one direction and then in the opposite direction, both sides of the stone may be covered and an even distribution obtained.

Mr. Collier.

The speaker has no objection to bleeding in a road. Any bituminous material which comes to the surface in the first hot weather may be covered with sand or stone chips (which will cost very little), and, if the bleeding has been uniform, a water-tight road will result. Two years ago, in Raleigh, N. C., a piece of bituminous macadam was constructed by the penetration method in front of the Union Station. This road was made with 2¼ gal. of asphalt, and, during the first summer, when it bled, sand was thrown over it to take up this excess of bleeding. Adjacent to this road there is a piece of sheet-asphalt pavement, and it is impossible for the casual observer to distinguish between the two surfaces. There has not been a particle of wear on this street, and it is in perfect condition to-day.

Mr. Connell.

WILLIAM H. CONNELL, ASSOC. M. AM. SOC. C. E.—In discussing bituminous pavements constructed by penetration and mixing methods, the speaker will confine himself to the experimental pavements on the White Plains Road, and endeavor to give an account of the present condition of the different pavements laid on this roadway, together with observations which he believes to be of sufficient importance to mention. The cost of repairs to each section is based on the construction cost per square yard of the respective sections, and the cost per square yard of repairs is obtained by dividing the total cost by the total number of square yards in the section. The construction cost is used for this purpose because it is deemed to be a fairer basis of comparison of the maintenance cost of the respective sections than

TABLE 6.—COST OF CONSTRUCTION AND REPAIRS.

Section No.	Square yards repaired.	Construction cost per square yard.	Total cost of repairs.	Maintenance cost per square yard for 1911.
1	6.4	$0.80	$5.12	$0.007
2	32.3	0.832	26.87	0.035
3	0.0	.....	0.00	0.00
4	0.0	.....	0.00	0.00
5	3.	0.75	2.25	0.003
6	3.5	.....	0.00	0.00
7	17.91	.....	0.00	0.00
8	8.0	0.124	8.99	0.019
9	2.7	1.33	3.59	0.009
10	54.6	1.077	58.80	0.072
11	243.2	0.051	12.53	0.051
12a*	354.1	.....	4.19	0.012
12b	5.8	0.654	3.79	0.032
12b*	118.0	.....	1.49	0.012
13	4.3	1.209	5.19	0.011
14	0.0	.....	0.00	0.00
15	413.2	0.04	16.67	0.040
16	35.	0.67	23.45	0.066
17	0.0	.....	0.00	0.00

* Surfaced with sand.

Mr. Connell. that of the actual cost of repairs, which in a number of cases would be misleading, owing to the high cost per square yard due to lack of proper facilities for making repairs. The cost data for repairs are given in Table 6.

WHITE PLAINS ROAD EXPERIMENTAL PAVEMENTS.

Section One: 0 + 00 to 3 + 25.—Bituminous pavement, mixing method, laid by the Barber Asphalt Company. Surface even. The mixture, however, is apparently too dry; 5.4% of asphaltic cement of 38 penetration was used. The penetration and percentage of asphaltic cement should be increased.

Section Two: 3 + 25 to 6 + 50.—Bituminous pavement, mixing method, laid in accordance with Borough of the Bronx Specifications. Surface even and entirely satisfactory. Repairs were due to moisture in the foundation which worked in from the railway tracks. There was no pavement between the railway tracks last winter.

Section Three: 6 + 50 to 8 + 00.—Surface even. An excess of asphaltic cement was used in the seal coat, which caused a slight bleeding during high temperatures. No repairs required.

Section Four: 8 + 00 to 11 + 25.—Bituminous pavement, mixing method, laid under the supervision of the Standard Oil Company. Surface uneven. Pavement laid during low temperatures; set before it received proper compression under roller; traffic increased unevenness until waves are about 2 ft. apart. Asphaltic cement apparently too soft at high temperature for this method of construction. No repairs required.

Section Five: 11 + 25 to 14 + 50.—Bituminous pavement, mixing method, laid under the supervision of The Texas Company. Surface even, but has settled under traffic more than that of any other section. Mixture not dense enough, ¾-in. stone only being used in same; has heaved in several spots, possibly due to moisture, as surface is not water-proof. Bled to some extent where seal coat was applied. Repairs due to settlement.

Section Six: 14 + 50 to 17 + 75.—Amiesite, laid under the supervision of the Amiesite Company. Surface satisfactory, but not as even as it was in June, 1911, possibly due to a combination of settlement of foundation and effect of traffic during high temperatures. Patch due to frost in foundation. Very slight bleeding during high temperatures.

Section Seven: 17 + 75 to 20 + 67.—Sicilian Asphalto, laid by the Sicilian Asphalt Company. Surface even. A very desirable surface for park drives and automobile traffic.

Section Eight: 20 + 67 to 22 + 75.—Bituminous pavement, mixing method, laid under the supervision of the Barrett Manufacturing Company. Surface somewhat wavy, apparently due to a combination

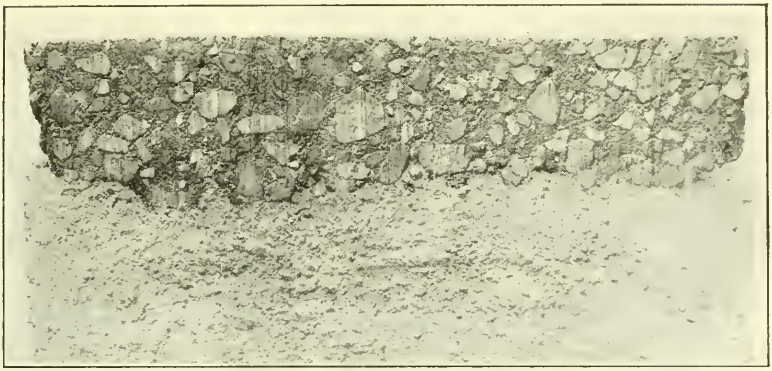


Fig. 10.—Sec. 1.—Bituminous Pavement, Built by Machine-Mixing Method. 73% Paving Gravel, 19½% Sand, 1½% Dust, and 5.4% Bermudez Asphalt.

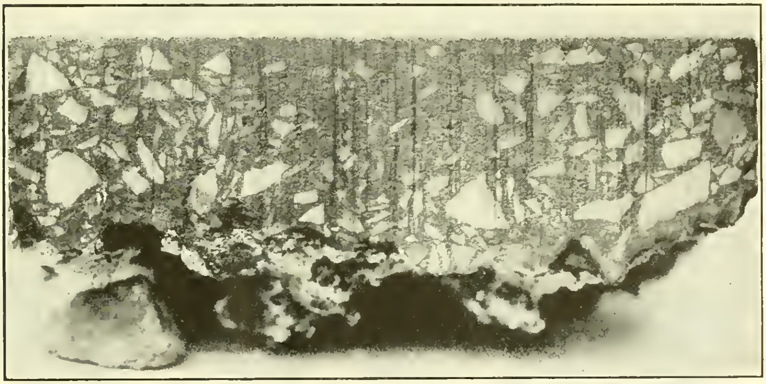


Fig. 11.—Sec. 2.—Bituminous Pavement, Built by Machine-Mixing Method, in Accordance with Borough of the Bronx Specifications. 2 Parts Trap Rock, Passing 1¼-in. Ring, with Maximum of 5% Dust, 1 Part Sand, and 7.4% Bermudez Asphalt.

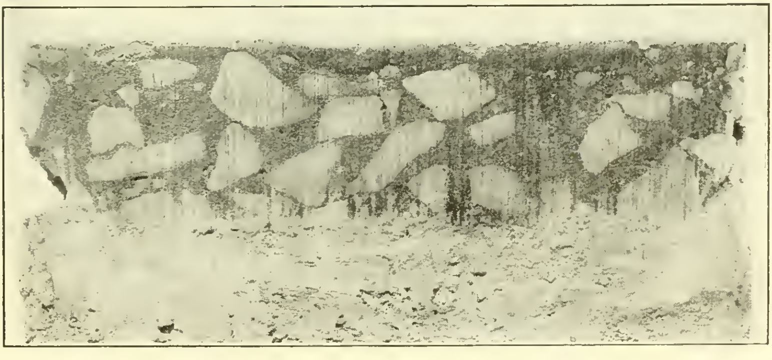


Fig. 12.—Sec. 3.—Bituminous Pavement, Built by Machine-Mixing Method. 2 Parts Trap Rock Passing 1¾-in. Ring, with Maximum of 5% Dust, 1 Part Sand, and 8% Bermudez Asphalt.

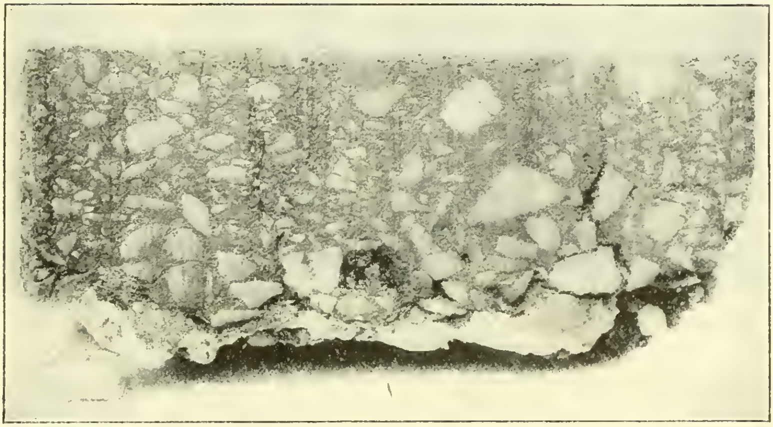


FIG. 13.—SEC. 4.—BITUMINOUS PAVEMENT, BUILT BY HAND-MIXING METHOD. 58% OF ¾-IN. TRAP ROCK, 20% OF ⅜-IN. CHIPS, 15% OF SAND, AND 7% OF STANDARD OIL COMPANY'S SPECIAL ASPHALT BINDER.

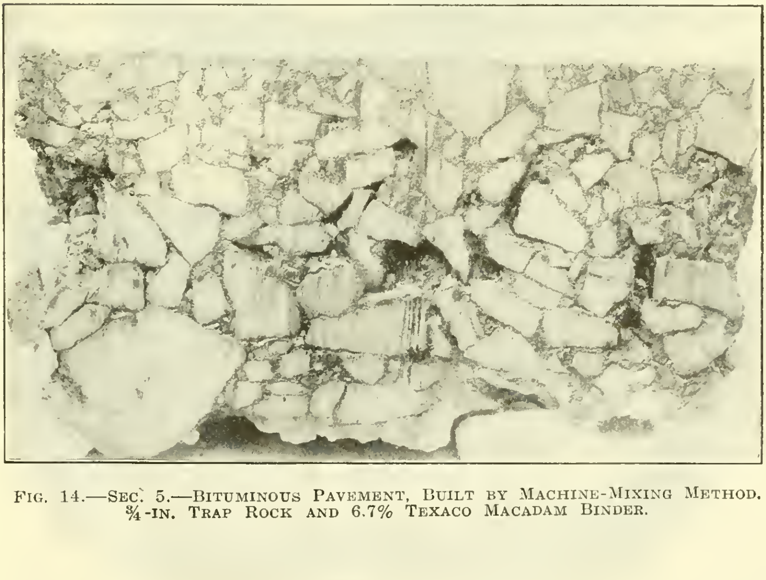


FIG. 14.—SEC. 5.—BITUMINOUS PAVEMENT, BUILT BY MACHINE-MIXING METHOD. ¾-IN. TRAP ROCK AND 6.7% TEXACO MACADAM BINDER.

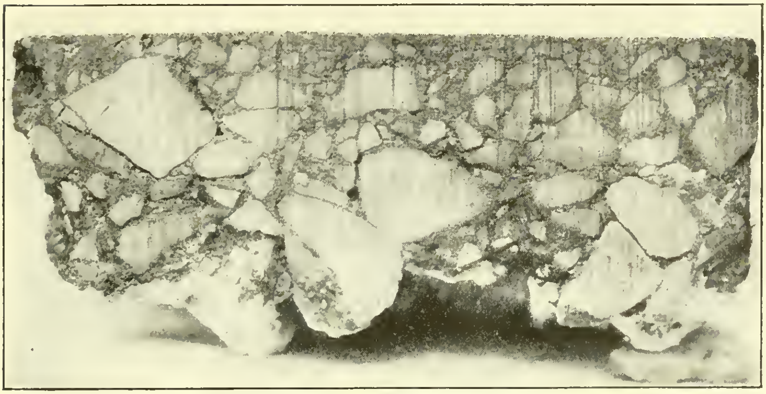


Fig. 15.—Sec. 6.—Bituminous Pavement Known as "Amiesite." Built by Machine-Mixing Method.

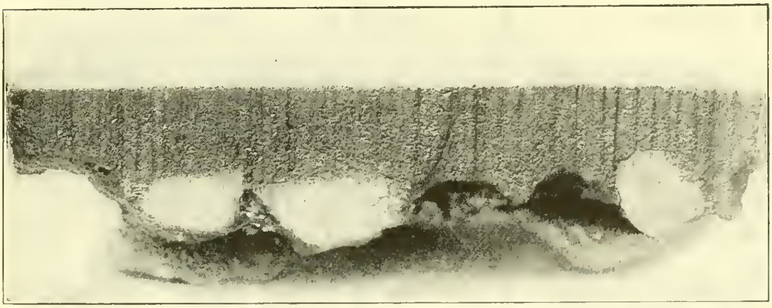


Fig. 16.—Sec. 7.—Bituminous Pavement, Built by the Sicilian Asphalt Company, by Machine-Mixing Method, Using "Asphalto" Paving Mixture.

Fig. 17.—Sec. 8.—Bituminous Pavement, Built by Hand-Mixing Method. Three Parts ¾-in. Trap Rock, 1 Part Chips, and 6½% Tarvia X.

Mr. Connell.

TABLE 7.

	Sec. 4. Standard Oil Special binder.	Sec. 5. Texaco Macadam binder.	Sec. 5. Texaco No. 55 Special Asphalt.	Sec. 8. Tarvia X.	Sec. 9. Bermudez Asphalt for 10% mixture.	Sec. 9. Bermudez Asphalt for 15% mixture.	Sec. 9. Tar and Asphalt 15%.	No. 9. Water-Gas Tar and Bermudez Asphalt 15%.	Sec. 10. Water-Gas Tar (from barrel).	Sec. 10. Water-Gas Tar (from kettle).	Sec. 11. Tarvia X.	Sec. 12. Sanford and Strain's Asphalt binder.	Sec. 15. Tarvia X.	Sec. 16. Bermudez Asphalt.	Sec. 17. Standard Oil Binder B.
Specific gravity 25° C./25° C.	1.014	0.9765	1.004	1.249	1.062	1.038	1.153	1.152	1.178	1.190	1.249	0.9943	1.246	1.037	0.9952
Penetration 25° C.	94	too soft	56	...	17	107	...	...	...	...	...	113	...	147	121
Float test, N. Y. T. L. Visc. 50° C.	...	...	...	1′ 38″	...	...	3′ 52″	4′ 36″	3′ 5″	52′ 44″	1′ 27″	...	1′ 21″	...	...
Loss at 163° C. 5 hours	0.00%	9.35%	0.19%	...	3.77%	3.80%	...	...	...	...	...	3.73%	...	3.11%	0.68%
Penetration residue 25° C.	72	192	45	...	8	43	...	...	...	...	...	62	...	64	74
Soluble in CS_2	99.64%	99.90%	99.71%	75.89%	95.34%	97.32%	97.84%	97.83%	97.83%	97.09%	76.27%	99.49%	76.07%	97.25%	99.24%
Organic matter insoluble in CS_2	0.32%	0.10%	0.13%	24.05%	2.67%	1.42%	1.86%	1.80%	2.09%	2.79%	23.49%	0.42%	23.84%	1.37%	0.68%
Inorganic matter insoluble in CS_2	0.04%	0.00%	0.16%	0.06%	1.99%	1.26%	0.30%	0.37%	0.08%	0.42%	0.24%	0.09%	0.09%	1.38%	0.08%
Bitumen insoluble in 86° Naphtha	22.52%	17.71%	24.74%	...	28.04%	22.32%	...	...	...	...	...	23.80%	...	22.53%	24.17%
Fixed carbon	15.55%	8.04%	10.97%	...	14.14%	12.25%	...	...	...	...	...	9.37%	...	12.05%	14.17%
Distillation by volume:															
Water	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
to 110° C.	...	...	...	0.2%	...	...	0.2%	0.1%	0.2%	...	0.5%	...	...	...	...
110° to 170° C.	...	...	...	2.5%	...	...	1.1%	0.5%	0.4%	0.3%	2.5%	...	1.3%	...	...
170° to 270° C.	...	...	...	18.8%	...	...	8.0%	8.4%	18.1%	6.9%	16.9%	...	19.5%	...	...
Pitch	...	...	...	78.5%	...	...	90.7%	91.0%	81.3%	92.8%	80.1%	...	79.2%	...	...
Distillation by weight:															
Water	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
to 110° C.	...	...	...	0.2%	...	...	0.1%	0.1%	0.1%	...	0.3%	...	...	...	...
110° to 170° C.	...	...	...	2.0%	...	...	0.8%	0.4%	0.3%	0.2%	1.9%	...	1.1%	...	...
170° to 270° C.	...	...	...	15.6%	...	...	6.8%	7.0%	15.3%	5.7%	14.1%	...	15.1%	...	...
Pitch	...	...	...	81.9%	...	...	92.0%	92.0%	83.8%	93.9%	82.9%	...	82.9%	...	...

Mr. Connell.

of settlement of foundation and effect of traffic during high temperatures. Slight bleeding during high temperatures.

Section Nine: 22 + 75 to 24 + 50.—Bituminous pavement, mixing method. Surface slightly wavy, possibly due to foundation. Asphalt has apparently helped tar, as this section did not show any tendency to break up in cold weather, as the tar did when used without the addition of asphalt. (See Section Ten.) Did not bleed to any appreciable extent during high temperatures.

Section Ten: 24 + 50 to 27 + 97.—Bituminous pavement, mixing method. (United States Tar.) Surface somewhat uneven, due to foundation. Binder very brittle in cold weather; possibly some repairs were due to moisture in foundation or overheated binder, but a num-

TABLE

	Sec. 1. Bermudez Asphalt.	Sec. 2. Bermudez Asphalt.	Sec. 3. Bermudez Asphalt.	Sec. 4. Standard Oil Special Binder.	Sec. 5. Texaco No. 55 Special Asphalt.	Sec. 5. Texaco Macadam binder.	Sec. 8. Tarvia X.
Specific gravity	1.063	1.061	1.049	1.020	1.005	0.979	1.254
Soluble in H_2O	......	......	......	......	......	......	......
Organic	0.09%	0.11%	0.13%	0.023%	0.02%	0.01%	0.27%
Inorganic	0.04%	0.05%	0.06%	0.023%	0.00%	0.03%	0.06%
Free carbon	3.94%	4.39%	0.05%	0.69%	0.57%	0.18%	23.56%
Ash	2.07%	2.77%	2.35%	0.04%	0.16%	0.09%	0.41%
Soluble in cold carbon tetrachloride	*	*	*	99.17%	99.10%	99.66%	......
Fixed carbon	18.84%	17.89%	17.82%	20.50%	15.86%	10.36%	39.40%
Paraffin	*	*	*	1.44%	0.28%	0.34%	......
Melting point of normal material	212° F.	215° F.	207° F.	194° F.	237° F.	139° F.	145° F.
Evaporation 5 hours at 170° C.	4.21%	3.62%	*	gain of 0.06%	gain 0.2%	7.84%	5.14%
Melting point of residue.	221° F.	217° F.	*	200° F.	237° F.	169° F.	174° F.
Penetration of residue at 4° C	2.2	2.6	*	4	8	23	6
Penetration of residue at 25° C	16.3	18.4	*	53	36	133	56
Evaporation 5 hours at 205° C.	7.35%	4.73%	1.96%	0.20%	0.78%	12.63%	9.27%
Melting point of residue.	237° F.	223° F.	211° F.	216° F.	258° F.	202° F.	187° F.
Penetration of residue at 4° C	1.6	2.6	4.5	4	5	17	2
Penetration of residue at 25° C	9.5	16.5	36.9	25	28	81	23
Solubility in 88° B. naphtha.	78.68%	69.17%†	78.65%	80.30%	59.21%	61.18%	......
Character of solution (oily or sticky)	sticky	sticky	sticky	sticky	sticky	sticky	......
Distillation:							
Up to 105° C	......	......	......	......	......	......	0.00%
105° to 170° C	......	......	......	......	......	......	0.00
170° to 225° C	......	......	......	...	......	......	0.09
225° to 270° C	......	......	......	......	......	......	1.21
270° to 300° C	......	.....	......	......	......	......	6.18
Viscosity 100° C.:							
Penetrometer	......	......	......	......	......	......	......
N. Y. T. L. Visc	1′ 15″	1′ 21″	1′ 24″	42″	1′ 43″	20″	19″
Viscosity 25° C.:							
Penetrometer	37.6	41	51.5	113.8	47.3	.....	......
N. Y. T. L. Visc	......	......	......	......	......	10′ 7″	32′ 50″

* Sample insufficient for these determinations.

ber were undoubtedly due to the brittle character of the binder in cold weather. This section, however, showed less tendency to bleed during high temperatures than the other sections on which tar was used for a binder. Mr. Connell.

Section Eleven: 27 + 97 to 29 + 00.—Barrett Manufacturing Company's "Modern Pavement." Surface even; put on an application of Tarvia B and sand, as there seemed to be a tendency for surface stone to separate. There was considerable moisture under the foundation when this pavement was laid; it bled to a considerable extent in places during high temperatures, due to excess of binder.

Section Twelve A: 29 + 00 to 30 + 50.—Bituminous pavement, penetration method. (Sanford and Strain's Asphalt Binder.) Surface

8.

Sec. 9. U. S. Tar and Asphalt 10%.	Sec. 9. Bermudez Asphalt for 15% mixture.	Sec. 9. Water-Gas Tar and Bermudez Asphalt 15%.	Sec. 9. Water-Gas Tar and Bermudez Asphalt 15% (mixed on work).	Sec. 9. Bermudez Asphalt.	Sec. 10. Water-Gas Tar.	Sec. 11. Tarvia X.	Sec. 12. Sanford and Strain's Asphalt Binder.	Sec. 16. Bermudez Asphalt.	Sec. 17. Standard Oil Binder B.	Sec. 18. Standard Oil Binder A.
1.159	1.044	1.185	1.172	1.072	1.193	1.251	0.999	1.035	1.076	0.988
......	......	......	......	......	......	......	......	......	......	......
0.045%	0.13%	0.08%	0.12%	0.10%	0.07%	0.54%	0.04%	0.11%	0.12%	0.08%
0.091%	0.09%	0.08%	0.04%	0.07%	0.07%	0.07%	0.04%	0.06%	0.00%	0.00%
4.96%	2.94%	3.54%	2.58%	6.47%	6.85%	24.26%	0.41%	3.10%	1.13%	0.35%
1.07%	0.14%	0.35%	0.32%	1.43%	0.08%	0.18%	0.11%	1.70%	0.16%	0.11%
56.62%	96.75%	70.28%	91.63%	92.95%	......	......	99.41%	96.58%	96.33%	99.41%
28.18%	18.54%	28.73%	26.65%	22.25%	33.96%	39.55%	11.80%	17.34%	22.10%	18.08%
0.03%	0.31%	0.70%	0.20%	0.55%	......	......	0.14%	0.45%	1.66%	1.85%
171° F.	194° F.	174° F.	165° F.	180° F.	199° F.	132° F.	191° F.	188° F.	177° F.	95° F.
8.14%	5.35%	3.56%	5.76%	3.47%	4.46%	6.05%	4.37%	4.73%	0.79%	0.08%
217° F.	228° F.	†191° F.	192° F.	206° F.	220° F.	158° F.	228° F.	218° F.	207° F.	122° F
0	3	1	0	5	0	4	7	5	20	84
9	8	26	24	30	4	111	16	11	189	soft
11.56%	8.63%	12.88%	19.16%	8.51%	8.07%	6.88%	8.73%	5.48%	3.18%	0.73%
237° F.	248° F.	214° F.	235° F.	226° F.	230° F.	178° F.	247° F.	220° F.	233° F.	129° F.
0	1	0	0	1	0	1	6	4	16	51
4	5.4	4	1.5	10	0	75	10	9	55	soft
59.21%	71.23%	31.06%	60.61%	57.86%	......	......	63.68%	79.24%	66.48%	88.27%
oily	sticky	oily	oily	sticky	......	......	oily	sticky	......	oily
0.00%	......	0.00%	0.00%	......	0.00%	0.00%	......	......	......	......
0.00%	......	0.00%	0.00%	......	0.00	0.00	......	......	......	......
0.00	......	0.24	1.27	......	0.00	0.00	......	......	......	......
0.00	......	0.14	7.16	......	0.67	7.92	......	......	......	......
1.13	......	0.54	0.56	......	0.50	4.23	......	......	......	......
......	......	......	......	......	......	......	......	......	......	......
45″	55″	36″	22″	49″	62″	24″	45″	41″	2′ 32″	18″
102.2	117.7	136.5	243	118	25.1	......	93.7	146	......	......
......	......	......	......	......	......	29′ 8″	......	......	soft	8′ 38″

† Used 86° B. naphtha.

Mr. Connell. slightly wavy; bled to a great extent during high temperatures, which necessitated an application of sand to take up excess binder; wearing surface is not as thick as was stated in report.

Section Twelve B: 30 + 50 to 31 + 00.—Standard Oil Company's Special Binder. Surface slightly wavy; bled to some extent, but not as badly as Section Twelve A. Spread sand to take up excess binder.

Section Thirteen: 31 + 00 to 33 + 00.—Hassam Concrete Pavement. Surface good, with the exception of two cracks extending the entire width of the roadway, and two places about 9 in. in diameter where the stone has picked up.

Section Fourteen: 33 + 00 to 34 + 50.—Water-Bound Macadam. This section is at present in an ideal condition for a suitable bituminous surface treatment. The top dressing has been scattered and washed away. It will be necessary to resurface this section next season or to apply a bituminous surface treatment.

Section Fifteen: 34 + 50 to 36 + 25.—Bituminous pavement, penetration method. (Tarvia X.) Surface even. Applied Tarvia B and sand, as surface stone seemed to be separating in places.

Section Sixteen: 36 + 25 to 37 + 75.—Bituminous pavement, penetration method. (Bermudez Asphalt.) Surface slightly wavy. No repairs made, thus far, except those due to a settlement of 1 ft. or more over a sewer; shows signs of moisture in the wearing surface in several spots, whether from the surface or foundation, is a question at present. Slight bleeding during high temperatures.

Section Seventeen: 37 + 75 to 39 + 22.—Bituminous pavement, penetration method. Surface even and very satisfactory to date. Slight bleeding during high temperatures.

Section Eighteen: 39 + 22 to 41 + 20.—Standard Oil Company's sand surface method. Bituminous surface has entirely disappeared. Surface was not in condition to receive treatment, owing to new patches where screenings had not set. The bituminous material applied to this section was not at all suitable for the traffic on this roadway.

In reference to the experimental pavements on the White Plains Road, statements have been made which are somewhat misleading, and, as it is the speaker's intention to make a report once a year, giving a full account of these experiments, accurate information may be obtained by referring to the *Transactions* of this Society. These experiments were made with a view to determine a suitable medium-priced pavement, and this was the reason for using, in many instances, a broken-stone foundation. When a statement is made to the effect that the surface is slightly uneven, due partly to the foundation, it does not necessarily mean that the foundation has failed or that the pavement has proved unsatisfactory. All the foundations are alike

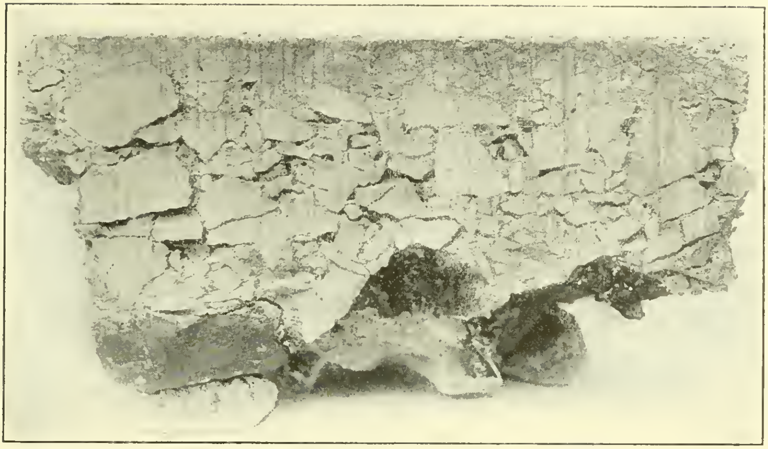


FIG. 18.—SEC. 9A.—BITUMINOUS PAVEMENT, BUILT BY HAND-MIXING METHOD. THREE PARTS ¾-IN. TRAP ROCK, 1 PART CHIPS, AND 7% MIXTURE OF TAR AND 15% BERMUDEZ ASPHALT, 110 PENETRATION.

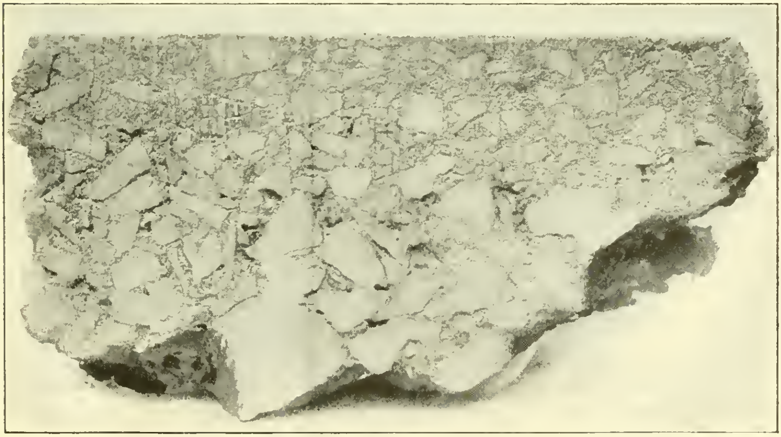


FIG. 19.—SEC. 9B.—BITUMINOUS PAVEMENT, BUILT BY HAND-MIXING METHOD. THREE PARTS ¾-IN. TRAP ROCK, 1 PART CHIPS, AND 7% MIXTURE OF TAR AND 15% BERMUDEZ ASPHALT, 60 PENETRATION.

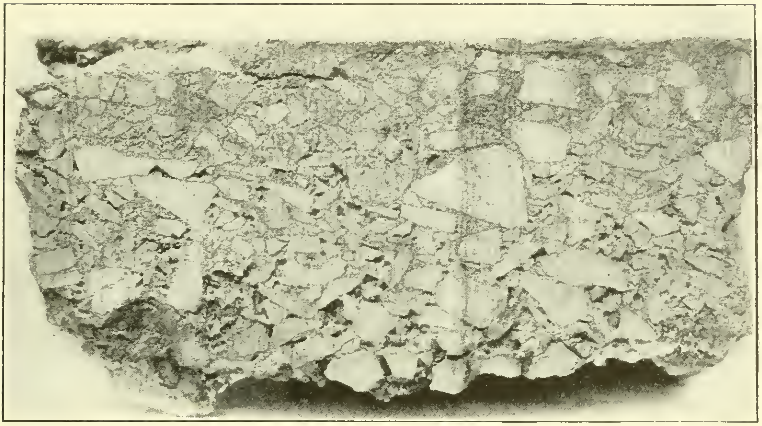


Fig. 20.—Sec. 9c.—Bituminous Pavement, Built by Hand-Mixing Method. Three Parts ¾-in. Trap Rock, 1 Part Chips, and 7% Mixture of Tar and 10% Bermudez Asphalt.

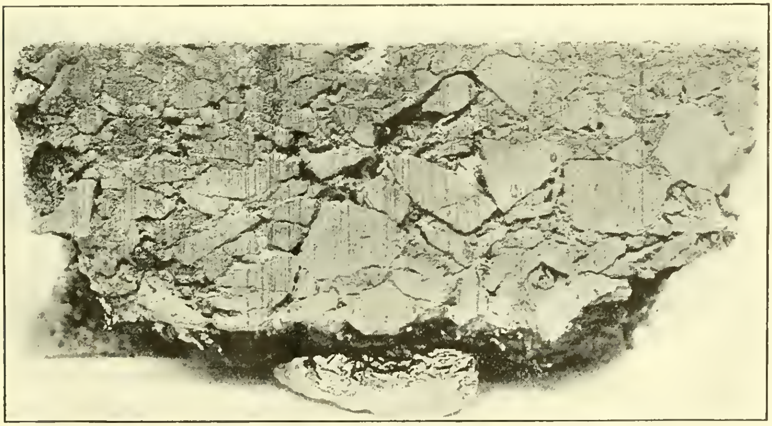


Fig. 21.—Sec. 10.—Bituminous Pavement, Built by Hand-Mixing Method. Three Parts ¾-in. Trap Rock, 1 Part ⅜-in. Trap Rock Chips, and 6% Refined Tar, Specification of U. S. Office of Public Roads.

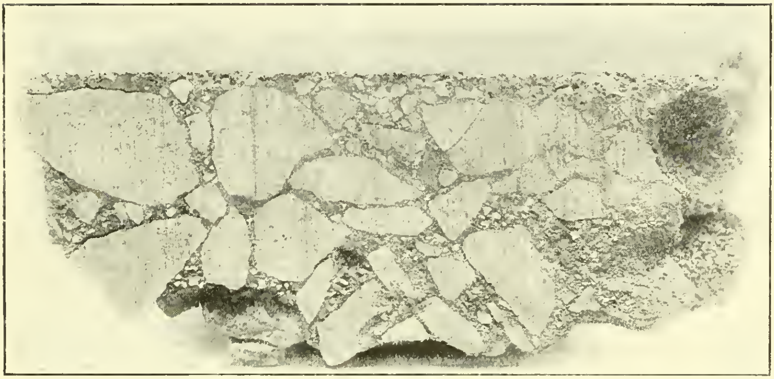


FIG. 22.—SEC. 11.—BITUMINOUS PAVEMENT, BUILT BY BARRETT MANUFACTURING COMPANY, USING PENETRATION METHODS. KNOWN AS "MODERN PAVEMENT."

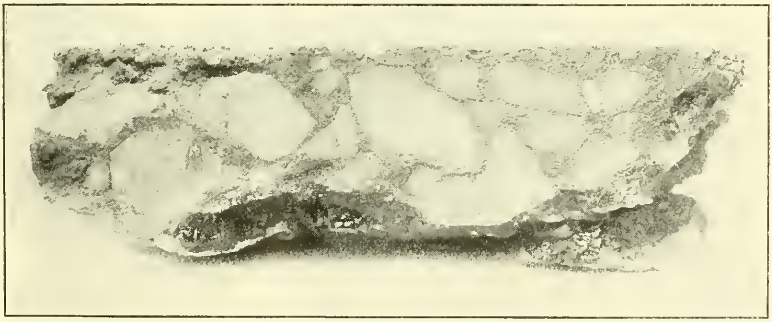


FIG. 23.—SEC. 12A.—BITUMINOUS PAVEMENT, BUILT BY PENETRATION METHOD. 1½-IN. TRAP ROCK, 2¾ IN. DEEP, ON WHICH 1½ GAL. STANDARD OIL COMPANY'S SPECIAL BINDER WAS APPLIED. LAYER OF ¾-IN. TRAP ROCK ROLLED, ON WHICH ¾ GAL. SPECIAL BINDER WAS APPLIED; COARSE SAND AND CHIPS SPREAD AND ROLLED.

Mr. Connell.

TABLE 9.

	Sec. 1.	Sec. 2.	Sec. 3.	Sec. 4.	Sec. 5.	Sec. 6.	Sec. 7.	Sec. 8.	Sec. 9, South.	Sec. 9, Middle.	Sec. 9, North.	Sec. 10.	Sec. 11.	Sec. 12, A.	Sec. 12, B.	Sec. 13.	Sec. 15.	Sec. 16.	Sec. 17.
Specific gravity 25° C./25° C.	2.341	2.402	2.392	2.508	......	2.679	2.222	2.565	2.574	2.524	2.562	2.514	......	2.550	2.560	2.575	......	......	2.670
Bitumen soluble in CS_2	5.3%	6.52%	6.56%	6.9%	5.7%	4.1%	7.6%	3.8%	5.2%	6.0%	5.7%	7.1%	3.2%	7.6%	8.0%	......	2.5%	7.2%	4.3%
Retained on 1½-in. screen	......	......	......	......	10.5%	14.2%	......	......	9.7%	......	......	......	12.7%	......	......	......	......	10.5%	23.0%
Retained on 1¼-in. screen	......	......	......	0.0%	7.2	8.4	......	0.05	12.1	......	......	......	15.7	0.0	30.5	......	12.0%	20.8	4.7
Retained on 1-in. screen	0.0%	......	0.0%	3.9	6.5	4.0	......	12.1	14.7	0.0	0.0	0.0	13.8	14.8	11.1	......	31.1	8.1	17.8
Retained on ¾-in. screen	1.7	0.0	5.9	4.6	20.7	7.0	......	19.4	14.7	3.5	26.5	2.9	17.8	17.9	14.7	......	26.2	19.3	11.4
Retained on ½-in. screen	6.0	10.7	13.0	14.5	23.2	16.9	......	28.1	9.0	18.8	15.0	17.5	14.1	19.2	14.3	......	11.0	14.3	9.9
Retained on ¼-in. screen	25.3	24.3	23.7	21.2	17.2	16.6	......	15.1	10.5	31.5	18.8	30.7	6.1	14.1	9.7	......	4.9	6.8	14.5
Retained on ⅛-in. screen	26.8	17.2	14.8	20.1	5.5	11.3	......	10.1	11.6	21.4	15.8	23.4	4.0	10.6	6.2	......	3.6	4.7	5.8
Passing ⅛-in. screen	......	......	......	......	9.2	21.6	......	15.2	17.7	21.8	23.9	25.5	15.8	23.4	13.5	......	11.2	15.5	12.9
Retained on 10-mesh screen	3.9	7.9	5.7	8.7	......	......	0.0	......	......	......	......	......	......	......	......	......	......	......	......
Retained on 20-mesh screen	3.5	5.2	3.6	9.1	......	......	5.8	......	......	......	......	......	......	......	......	......	......	......	......
Retained on 30-mesh screen	2.8	2.7	2.5	3.6	......	......	16.1	......	......	......	......	......	......	......	......	......	......	......	......
Retained on 50-mesh screen	8.8	9.8	10.0	3.8	......	......	33.9	......	......	......	......	......	......	......	......	......	......	......	......
Retained on 80-mesh screen	10.0	12.0	12.1	2.9	......	......	21.3	......	......	......	......	......	......	......	......	......	......	......	......
Retained on 100-mesh screen	3.8	3.5	3.3	0.9	......	......	5.0	......	......	......	......	......	......	......	......	......	......	......	......
Retained on 200-mesh screen	4.3	3.4	2.8	2.0	......	......	7.8	......	......	......	......	......	......	......	......	......	......	......	......
Passing 200-mesh screen	3.1	3.3	2.6	4.7	......	......	10.1	......	......	......	......	......	......	......	......	......	......	......	......

Mr. Connell.

with the exception of two which are of concrete. The subsoil conditions in all sections are about the same. In one section the speaker noticed considerable moisture in the subsoil. It was also noticed that where the hardest asphalt cements were used, the surface did not push or wave as badly as with the softer grades, the foundation conditions being the same. As the method of laying the stone was the same in each case, it further strengthens the speaker's belief that the wavy condition is due entirely to differences in the consistencies of the materials. The mistake should not be made of comparing these second-class pavements with a first-class pavement laid at a much greater cost. In the discussion on these experiments the economic point of view has been lost sight of in some instances and comparisons with the ideal have been made rather than with a second-class pavement.

Tables 7, 8, and 9 contain analyses of the bituminous materials used in the construction of the different sections. The tests given in Table 7, were made by the U. S. Office of Public Roads in accordance with its methods for testing bituminous materials. In Table 8 are given the results of tests on the same materials made by the New York City Testing Laboratory in accordance with the methods proposed by the Special Committee on Bituminous Materials for Road Construction of this Society. Table 9 is a report of the U. S. Office of Public Roads, and gives the density of pavements, percentage of bitumen extracted, and mechanical analysis of the mineral aggregate of sections of the respective pavements which have been subjected to traffic for 11 months. A traffic census was taken during 4 days each month from 5 A. M. to 9 P. M., from January, 1911, to and including January, 1912. Table 10 gives the daily average of the 4 days' traffic for each class.

TABLE 10.—TRAFFIC CENSUS.

	Jan.	Mar.	Apr.	May.	June.	July.	Aug.	Oct.	Nov.	Dec.	Jan.
Horses, without vehicles	2	3	2	3	4	2	6	2	8	2	1
Horse vehicles, light	31	45	39	35	27	36	48	34	31	24	10
Horse vehicles, heavy	38	50	73	66	70	71	45	72	75	46	44
2-horse vehicles, light	5	8	8	6	3	2	34	8	2	2	10
2-horse vehicles, heavy	35	46	76	80	77	71	73	76	81	52	31
3-horse vehicles	1	1	2	1	1	1	1	1	1	1	1
4-horse vehicles	1	..	1	1	1	..	1	1	1	1	2
Motorcycles	2	4	9	11	6	9	3	6	8	5	1
2-passenger cars	5	11	14	19	18	15	15	17	20	10	3
4- and 5-passenger cars	28	60	72	88	94	69	45	57	75	45	10
6- and 7-passenger cars	16	38	57	52	41	53	47	83	81	68	30
Motor trucks	14	23	24	28	31	32	27	45	48	37	39
Traction engines	..	1	..	..	..	1	..	..	..	..	..
Miscellaneous heavy traffic	..	1	..	..	..	..	..	..	..	..	..
Totals	178	291	377	390	373	362	345	402	431	293	182
Totals, horse-drawn vehicles	111	150	199	189	179	181	202	192	191	126	98
Totals, motor vehicles	65	138	176	198	190	179	137	208	232	165	83

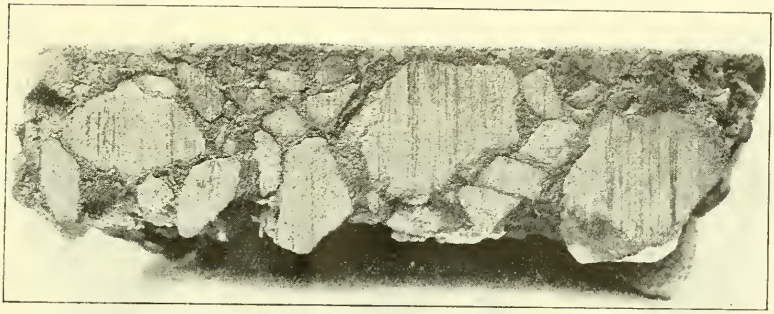


FIG. 24.—SEC. 12B.—BITUMINOUS PAVEMENT, BUILT BY PENETRATION METHOD. SAME CONSTRUCTION AS SEC. 12A, EXCEPT THAT SANFORD AND STRAINS ASPHALT BINDER WAS USED.

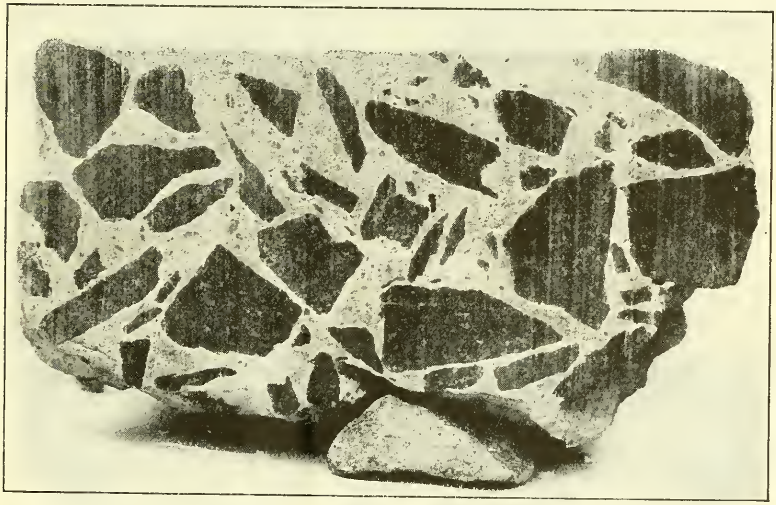


FIG. 25.—SEC. 13.—"HASSAM" CONCRETE PAVEMENT, BUILT BY FORCING INTO VOIDS OF STONE BY PRESSURE MACHINE, MORTAR OF 1¾ PARTS SAND TO 1 PART PORTLAND CEMENT.

FIG. 26.—SEC. 15.—BITUMINOUS PAVEMENT, BUILT BY PENETRATION METHOD. 1½-IN. TRAP ROCK, ROLLED TO A DEPTH OF 2½ IN., 2.3 GAL. TARVIA APPLIED. LAYER OF CHIPS ROLLED, ¾ GAL. TARVIA APPLIED. FINISHED BY ROLLING LAYER OF CHIPS.

FIG. 27.—SEC. 16.—BITUMINOUS PAVEMENT, BUILT BY PENETRATION METHOD. 1½-IN. TRAP ROCK, ROLLED TO A DEPTH OF 3 IN., 1½ GAL. BERMUDEZ ROAD ASPHALT APPLIED, LAYER OF CHIPS ROLLED, ½ GAL. BERMUDEZ ASPHALT APPLIED, OVER WHICH A LAYER OF CHIPS WAS ROLLED

FIG. 28.—SEC. 17.—BITUMINOUS PAVEMENT, BUILT BY PENETRATION METHOD. 1½-IN. TRAP ROCK ROLLED TO A DEPTH OF 6 IN., 1⅓ GAL. STANDARD OIL COMPANY'S BINDER B APPLIED, LAYER OF ¾-IN. TRAP ROCK ROLLED, 1.1 GAL. BINDER B APPLIED, AFTER WHICH LAYER OF CHIPS WAS ROLLED.

T. HUGH BOORMAN, ESQ.—The speaker believes that there are sections throughout the United States where natural rock asphalt could be used economically. This material has been tested longer in road construction than any other bituminous surfacing. Mr. Boorman.

In 1874, or 1875, Pennsylvania Avenue, in Washington, D. C., was constructed partly with European rock asphalt. About 1895, 110th Street, from Columbus to Seventh Avenues, and Convent Avenue, from 145th to 146th Streets, in New York City, were laid with natural rock asphalt, and are in splendid condition at the present time. As the repairs have been made with ordinary sheet-asphalt mixture, the cost of maintenance on a rock asphalt street, if repaired with that material, is not known.

Among the twenty-three different trial methods of road surfacing, laid under the supervision of the Road Board of England, on the Folkestone-London Road, in the County of Kent, in the summer of 1911, and which the speaker had the privilege of observing, through the courtesy of Col. Crompton, was a section of natural rock asphalt broken into rectangular pieces, spread on an old macadam road about 6 in. thick, and then rolled until compressed to a thickness of about 4½ in.

A. W. DOW, ESQ.*—In regard to the experimental pavements on the White Plains Road, in the Borough of the Bronx, it has been stated that, in some instances, the foundations were responsible for the present poor condition of the wearing surface. Mr. Dow.

The wavy surfaces of some of these experimental pavements have been attributed to the use of bituminous material which was too soft at high temperatures, but the speaker believes it to be more likely that this waviness was caused by the poor foundation, as the softness of the material is very seldom the direct cause of the waving of pavements. Many instances might be mentioned in which excessively soft materials have been used in pavements, and the surfaces have not waved in the slightest. One of the most striking of these instances was in Washington, where a great many coal-tar pavements were laid at one time. Many of these pavements were laid very soft, but one on O and 21st Streets was so soft that traffic had to be kept off it for 3 months during the first two summers of its existence, because horses' hoofs picked out large pieces of the surface and wagons left deep ruts. This pavement never rolled in the slightest, and, although it was laid in the early Eighties, it is in fairly good condition to-day. In fact, the speaker has never seen a soft coal-tar pavement wave where the foundation was properly constructed, excepting, of course, on the side of hills. Coal-tar is one of the most susceptible materials

* Consulting Chemist, New York City.

Mr. Dow. to changes in temperature known and, therefore, becomes very soft in the heat of summer.

This same statement is true of asphalt pavements, which may be illustrated by another pavement in Washington, at the intersection of 13th Street and Massachusetts Avenue. This work was an old coal-tar pavement re-surfaced with 1½ in. of sheet-asphalt pavement laid directly on the smooth surface of the coal-tar. This thin sheet of asphalt contained 14% of bitumen which had a penetration of more than 120 when tested by the speaker at least 15 years after it had been laid. It is more than probable that the asphalt used in this wearing surface had more than 200 penetration when first laid. This coating of wearing surface did not adhere to the coal-tar pavement, and in hot weather traffic left deep indentations in passing over it, yet in no instance did this surface wave.

The speaker might also cite the case of oiled roads, many of which he has seen in California. These were composed of thick Maltha, fluid at ordinary temperatures, and sand. They could easily be dug out with a knife, yet on good foundations there was no sign of waving.

Still another most common example is the binding together of sand on sandy beaches by water. As long as the voids of the sand are filled with water, the surface remains hard, and where the sand is fine it makes a most excellent roadway. An example of this is the beach at Ormond, Fla., where so many automobile speed contests have been carried on.

Conclusions as to the value of various bituminous materials drawn from experimental pavements are useless unless the foundations in all cases are impervious and rigid, and all other factors which go to make up the pavement are identical. If in one experimental patch a foundation should give way slightly, a wavy condition or a breaking up of the wearing surface starts immediately. This condition will spread rapidly, no matter whether the bituminous binder is hard or soft. Of course, a soft bituminous binder will wave much more readily than a hard one when it has once started.

There is one other fact which cannot be too strongly impressed on engineers in the building of bituminous roads, namely, that all bituminous-bound roads require more carefully constructed foundations than water-bound macadam roads. This is because water or moisture cannot dry out of a road through its surface when it has been coated with bitumen, and, therefore, such a road is entirely dependent on some means of drainage.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1227

REBUILDING THREE LARGE PUMPING ENGINES.*

By Charles B. Buerger, Assoc. M. Am. Soc. C. E.

The Queen Lane Pumping Station, at Wissahickon, on the Schuylkill River, Philadelphia, was equipped in 1895 and 1896 with four single-acting, vertical, triple-expansion, crank-and-flywheel, pumping engines, built and installed by the Southwark Foundry and Machine Company, of Philadelphia. Some of the principal dimensions of these engines were as follows:

Rated capacity	20 000 000 gal. per day.
Speed	22 rev. per min.
Stroke	54 in.
Plunger, diameter	34½ in.
High-pressure cylinder, diameter	37 in.
Intermediate-pressure cylinder, diameter	62 in.
Low-pressure cylinder, diameter	96 in.
Piston rod, diameter—two to each cylinder	5¼ in.
Distance rod, diameter—4 to each pump	5 in.
Main journal, diameter	18 in.
Main journal, length	28 in.
Crank-pin, diameter	12 in.
Crank-pin, length	9 in.
Cross-head pin, diameter	13 in.
Cross-head pin, length	10 in.
Flywheel, diameter—two to engine	17 ft. 9 in.
Suction and discharge pipes	48 in.

* Presented at the meeting of March 20th, 1912.

Valve area per suction deck............801 sq. in.
Valve area per discharge deck...........801 sq. in.
Total water pressure...................246 ft.
Steam pressure........................150 lb. per sq. in.

The contract price for the four engines in place was $294 148, which is low when compared with the present-day cost of four engines of the same capacity; but, taking into account the difference in weight and materials, the price was very fair.

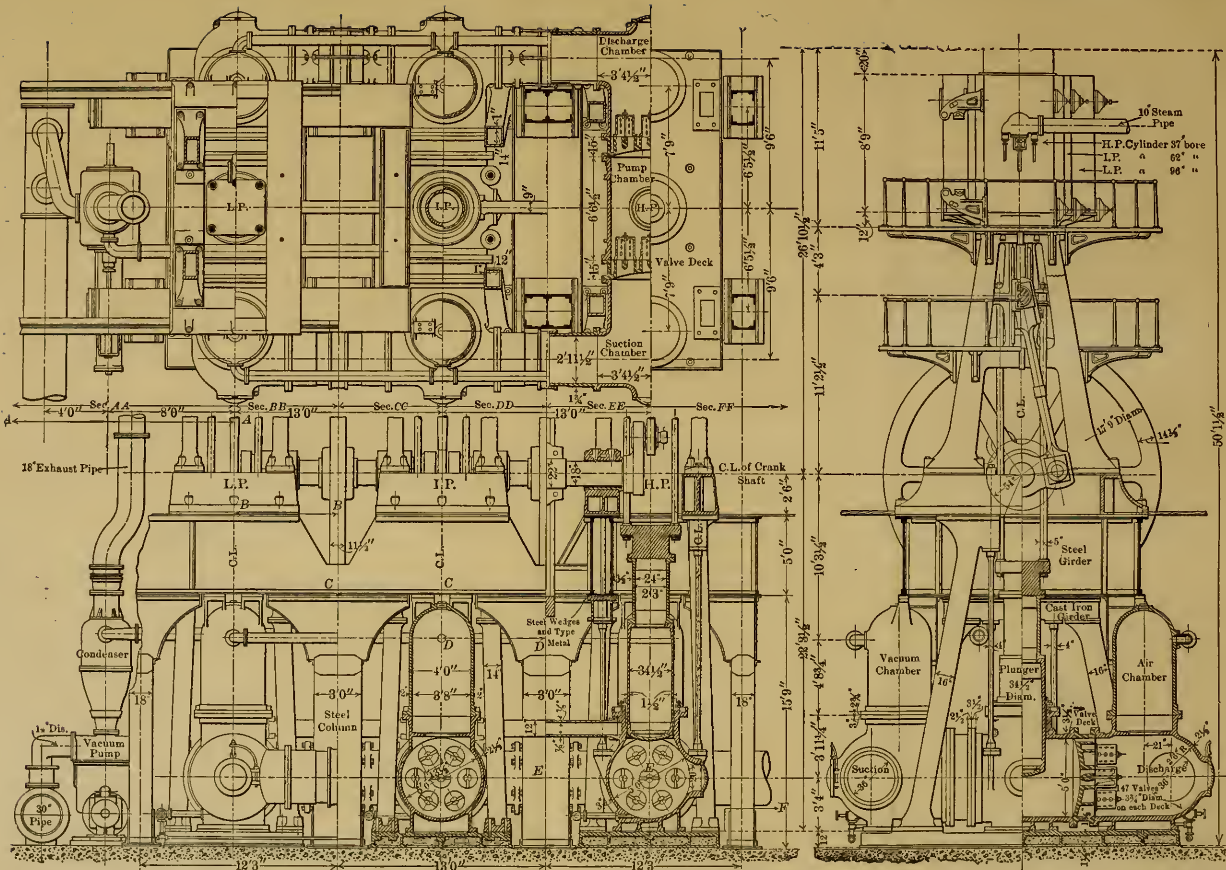
The design, in some respects, was in accordance with accepted practice, and in other respects unusual. The steam end was of the double A-frame type, built generally, as is common to-day, except that the steam distribution valves, instead of being of the usual Corliss construction, were of the gridiron design, and operated through a cam motion by eccentric rods on the main shaft. The steam end was carried from the basement floor by a structural steel framework, consisting of eight box-columns in two lines, carrying two longitudinal plate girders, and four transverse, double-web box-girders, one under each main bearing. The arrangement of the steam end and supporting steel structure is shown on Plate XI, which is a drawing of the rebuilt engine.

The pump end under each steam cylinder consisted in effect of three cylindrical chambers, placed one over the other, the lowest carrying the suction valve deck, the middle one an annular flat discharge valve deck, and the upper one a stuffing-box for the vertical plunger. The three sections were bolted together, the lowest to the foundation and the top one tied to the steam bed-plate by four steel bolts.

The pumping engines were unsatisfactory in operation from the time of earliest service. The structural steel framework was not braced to give sufficient rigidity to the steam end, and the vibration rapidly wore out all the wearing parts, resulting in misalignment and frequent breakages. The water end was even less satisfactory; the shapes of the castings were unsuited to the service, and some as thick as 4 in. lasted but a short time; the pumps, too, were insufficiently braced, vibration made the joints leak, and air reduced the pump deliveries.

The four engines were kept in regular but inefficient service, at the cost of heavy repairs, until 1907, when the introduction of a filtered water supply from another city plant permitted a partial shut-down

PLATE XI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXV, No. 1227.
BUERGER ON
REBUILDING THREE LARGE PUMPING ENGINES.



SCALE OF FEET
0 1 2 3 4 5

QUEEN LANE PUMPING STATION
REBUILT PUMPING ENGINE
GENERAL PLAN

of the station. In 1908 it was decided to build an additional filter station at the Queen Lane Reservoir, which would demand a regular supply of 70 000 000 gal. per day; the location of the proposed filter station, its elevation, and the pipe connections already in service to the Queen Lane Pumping Station, called for a water supply from this place, and it was necessary to make changes to put the machinery into effectual operation. New engines were estimated to cost, erected, about $100 000 each, and these could be expected to give a duty of 155 000 000 ft-lb. per 1 000 000 B. t. u. with the available steam pressure of 140 lb.

The reconstruction of the old engines was estimated to cost $175 000 for the four engines, about $44 000 each; they would then be capable of giving a duty, based on their performance in 1896, of about 140 000 000 ft-lb. per 1 000 000 B. t. u. The saving in installing new and more economical machinery would be, in round numbers, $2 000 per year in fuel—insufficient to warrant the additional expenditure. The plan of retaining the old machines in service and rebuilding them was accordingly adopted.

The alteration of any completed and used work is always full of complications and difficulties, markedly so where used machinery is in question. The common course in such case is to make a contract for a lump sum for the completed reconstruction, according to specifications based on the results sought. The writer, in charge of this work for the Philadelphia Bureau of Water, found this course, though easy, highly objectionable. Primarily, there could be no competition for work of this character, as manufacturers do not care to compete with the original builder of an engine for such a contract, so that bidding in this way means but a single proposal, and this cannot be expected to be a low one. Then, the difficulties of estimating on such work intelligently are considerable, and a losing contract is a temptation to the contractor to slight the work. The writer has a case in mind where a reputable manufacturer undertook, at an apparently generous price, to rebuild one of his old engines; after he had done some work on it and was able to appreciate what he had before him, he offered to abandon the old material entirely and to furnish in place an entirely new machine of the same capacity at his bid price. Lastly, and of most importance, it is impracticable to foresee exactly what will be required, and to call for the items explicitly, and if dependence is placed on broad specifications, disputes will arise in the execution of the work which cannot easily be adjusted equitably.

The unit-price contract presented the only plausible alternative.

Specifications.

The work was advertised on a brief specification for one engine. The following are the more important paragraphs:

(*1*) *Work to be Done.*—The work to be done consists of furnishing and delivering the material for rebuilding the pump ends of one vertical pumping engine at the Queen Lane Pumping Station, together with all appurtenances, and the delivery of one air pump.

(*2*) *Contract Plans.*—The following plans show the general design to be followed: Sheet No. 1.—Side elevation and sections; Sheet No. 2.—End elevation and sections.

(*3*) *Detail Plans.*—The City will furnish all detail plans for the work.

(*4*) *Patterns.*—The Contractor shall build all necessary patterns for the entire work; these patterns shall become the property of the City, and, on the completion of the work shall be delivered to the City shops. All flange drilling shall be done from approved jigs, which shall likewise become the property of the City, without extra charge.

(*5*) *Basis of Award.*—The following quantities shall be used in comparing bids:

Item	1-*a*	Cast iron—all kinds	600 000 lb.
"	1-*b*	Cast steel—all kinds	35 000 "
"	1-*c*	Forgings, steel and wrought iron	35 000 "
"	1-*d*	Bolts, nuts, and washers	15 000 "
"	1-*e*	Composition—all kinds	6 000 "
"	2	One air pump, delivered	Lump sum.

(*6*) *Character of the Work.*—All castings, forgings, etc., shall be machined to the dimensions shown on the detail plans, and in finish shall equal the present best practice. All work shall be fitted together and erected in the shop.

(*9*) *Variation of Weights.*—Payment for castings will be made on the basis of actual weights delivered; except that the weight paid for shall in no case be more than 5% in excess of the calculated weight. Castings weighing less than within 8% of the calculated weight, or at any point not within 10% of the designed thickness, will be rejected.

The following are the specifications for the materials:

(*19*) *Painting.*—All castings and other details shall be inspected and approved before painting. All metal work not finished shall receive two coats of paint. All finished surfaces shall be coated with white lead and tallow.

(*20*) *Shop Test.*—All castings shall be subjected to a shop test of 300 lb. per sq. in., hydraulic pressure.

(*21*) *Time of Completion.*—The Contractor shall begin work under this contract within 10 days from the time of notice to begin work, and shall prosecute the work with diligence, and to the satisfaction of the Engineer.

(*22*) *Payments.*—Payments, in all cases, will be made at the unit prices bid, which are to include the delivery of the material in the Queen Lane Pumping Station, ready for final erection.

The contract for this engine was awarded to the I. P. Morris Company, of Philadelphia, at the following prices:

Cast iron	$0.0440	per lb.
Cast steel	0.0800	" "
Forgings	0.0825	" "
Bolts, nuts, and washers	0.0850	" "
Bronze and composition	0.5600	" "

Work was begun on May 22d, 1908; the first engine was completed and put into service on July 14th, 1909, a total elapsed time of 418 calendar days. Two additional engines have been similarly rebuilt, the time required for each being approximately one year.

The costs for the three units have varied somewhat, particularly for the repair work on the steam end. The average cost for one engine is approximately as follows:

Cast iron	450 230 lb.	$19 810.12
Cast steel	45 430 "	3 634.40
Forgings	9 596 "	791.67
Bolts, etc.	18 596 "	1 580.66
Composition	3 496 "	1 957.76
Steam end repairs, including material and labor.		11 000.00
Air pump		1 370.00
Erection, labor (estimated)		4 000.00
Total		$44 144.61

Design of the Pump.

Plate XI shows the general assembly and some details of the construction.

Framework and Columns.—The structural steel framework supporting the old steam ends was inspected carefully and found to be in excellent condition; the rivets, with few exceptions, were tight, and the joints solid. The vibration to which it had been subjected, while severe enough to disable the machinery, had left the steelwork uninjured. As a measure of economy, the steel frame was retained in place, loose rivets were replaced, and the columns were solidly bolted down and grouted. Under each steel box-girder supporting a main bearing was placed a pair of cast-iron columns, battered in two directions, resting on the pump bed-plate at the bottom, and at the top connected by a cast-iron strut and girder. This cast-iron girder was erected $\frac{1}{2}$ to $\frac{3}{4}$ in. below the old steel girder, and was separated from it by steel wedges, 2 in. wide, at about 12-in. centers, driven into place solidly. The top flange of this girder was bolted to the lower angles of the steel by 1-in. bolts 12 in. apart, and the space between the two was filled with type metal.

As the columns were needed only as a bracing, they were made as light as it was practicable to cast them, of box section, 1 in. thick for the short columns, and $1\frac{1}{4}$ in. thick for the long columns, at the outside supports.

The cast-iron girders weigh 3 000 lb. each, the short columns 3 600 lb., and the long columns 6 900 lb.

Base-Plates.—A cast-iron base-plate was placed under each pump end. The base-plate has no structural value, and is of little value, except as warranted by the requirements of convenience in erection. It permits of locating and leveling the members, once for all; that is, four bracing columns, the pump chamber, and the suction and discharge chambers. It is of such a size as to connect all these, 12 in. deep, and generally $1\frac{1}{2}$ in. thick. Both top and bottom surfaces are planed. The weight of each plate is 24 000 lb.

It is interesting to note that the first plate cast showed a bend of $1\frac{1}{2}$ in. in cooling, the rough casting having the middle of the length that much lower than the line connecting the two ends. A bend in the pattern corrected this in subsequent castings.

Arrangement of Pump Ends.—The pump end is arranged in what is commercially known as the direct-flow type; the suction valves are on one side and the discharge valves at the other side of the pump chamber. The name is to some extent deceiving, as there is no reason to believe that the actual flow of the water is any more direct, or that the hydraulic resistance is even a little less, than in other arrangements not so well favored in name. This arrangement, however, permits a very satisfactory location of suction and discharge air chambers, both directly at the connection with the pump chambers. The writer considers this position of air chambers of decided advantage, and adopted the direct-flow type for this reason.

Pump Chambers.—The pump chamber is a T-shaped casting, 60 in. in diameter, and 3 in. thick. The corresponding stress in the body of the chamber is 1 000 lb. per sq. in., under working conditions. The allowed stress is apparently low, but is necessitated by the condition of intermittent and regularly repeated variations of pressure from a little lower than the suction, to a little higher than the discharge, pressure.

A comparison with some other pumps is made in Table 1.

TABLE 1.—COMPARISON OF PUMPS.

No.	Builder.	Date.	Style.	Capacity, in millions of gallons per day.	Working stress in pump chamber.
1	Southwark Foundry and Machine Company	1894	Horizontal, double-acting.	12	1 110
2	Henry R. Worthington	1901	Duplex, horizontal, double-acting	5	1 260
3	Holly Manufacturing Company..	1901	Horizontal, double-acting, Gaskill	10	1 370
4	Holly Manufacturing Company..	1907	Vertical, triple, single-acting	20	1 100
5	Allis-Chalmers Company	1907	Horizontal, double-acting.	6	1 100
6	Snow Steam Pump Company	1908	Horizontal, double-acting.	7½	1 370
7	Bethlehem Steel Company	1909	Horizontal, double-acting.	15	2 100 (cast steel)
8	City of Philadelphia	1909	Vertical, triple, single-acting	20	1 000

The critical points in a pump chamber are at the outlet openings for heads, manholes, and plungers. Where possible, it is desirable to round these with a curve of long radius, and also to thicken the metal, in order to get the necessary resistance. This detail is shown in the longitudinal section of the pump chamber. Where such construction is undesirable because of the formation of air pockets, as at the plunger outlet of a vertical pump, a shrink bolt can be added to advantage. In this design, a 3½-in. steel bolt is used at each side of the plunger opening, tightened up when hot, and stressed in cooling to its elastic limit.

Any exact determination of the stresses at such a corner is not to be expected, as these will vary from a maximum at the corner to a low figure some distance away; and it often happens that, even when the average stress through the section figured as a whole is very moderate, the shapes of the casting are such that unreasonably high stresses are imposed locally, and the casting will fail, though there is a sufficient weight of metal to do the work if it were disposed more judiciously.

For the purpose of comparison, the writer assumes that a stress figure can be obtained, by assuming that the corner forces are exerted uniformly over the length of curved section, and extending beyond the curve a length of three thicknesses of metal. Where a flange intervenes in this distance of three thicknesses, the metal outside the body line is not included. It is recognized that the maximum stresses at some points may be several times the quantity thus obtained. The method gives credit to easy curves, lower average stresses being obtained for curves of long radius; and conversely, straight sections are assumed to reinforce to only a limited extent, the length being proportionate to the thickness; flanges are supposed to be useful only for resisting the bolt strains. Qualitatively, these assumptions are undoubtedly correct, though the weight given is inexact.

On this basis, a comparison of the stresses in some of the pumps in Table 1 is given in Table 2.

TABLE 2.—Comparison of Stresses in Pump Cylinders.

Reference number, from Table 1.	Builder's name; and size of pump.	Corner stress, in pounds per square inch.
4............	Holly—20 000 000 gal. per day........	1 840
5............	Allis—6 000 000 " " "	930
6............	Snow—7 500 000 " " "	1 640
7............	Bethlehem—15 000 000 gal. per day....	2 100
8............	Philadelphia—20 000 000 gal. per day (neglecting shrink bolt)............	1 260

While this method of calculation is arbitrary, it is of some value when taken in connection with Table 2, which shows some successful modern practice. The writer is familiar with many examples in practice which, on the standard of computation here given, show abnormally high stresses; he is also familiar with many cases of failure of such examples; and one failure is more instructive than a multitude of apparent successes. The weight of the pump chamber is 20 000 lb.

Stuffing-Box.—The stuffing-box is of bronze, fitted in the customary manner. The first packing used was hemp rings, $1\frac{1}{8}$ in. square, placed in single rings and separated by brass spacers to keep the packing in proper shape. This is the most common form for pumps of this type, and is satisfactory for clean water. For the muddy and gritty water pumped at this station, the hemp packing was a failure. Good service has been obtained since by a semi-metallic packing consisting of wedge-shaped, white metal, wearing rings, held in place against the plunger surface by hemp fillers between. This packing has the particular advantage that it has no tendency to hold grit against the plunger surface and thus cut it.

Connecting Bolts.—The connecting bolts for the flange joints are $1\frac{1}{2}$ in., faced under the heads and under the nut, in spot-faced holes, $1\frac{19}{32}$ in. in diameter, spaced generally at about 5-in. centers. The bolts are designed for a maximum stress of 5 000 lb. per sq. in.

Gaskets.—The gaskets are of canvas, sewed into rings 3 in. wide, and painted with coal-tar. The flange joints are male and female. It is rather more common to use for this service gaskets made of rings of heavy paper soaked in linseed oil or red lead. This latter gasket is sufficient where the surfaces are perfect and the bolts are conscientiously tightened up until solid; but, where any surface may be slightly imperfect, or the bolts may be allowed to loosen, the tarred canvas joint is safer, as it will allow some water to blow through until the leak is detected, without destroying the gasket and compelling replacement.

Tension Rods.—The pressure of the plunger against the pump chamber is carried directly back to the steam end bed-plate by four steel tie-rods, 4 in. in diameter, giving a working stress of 2 000 lb. per sq. in. A low unit stress is here necessary to prevent a yield which would transfer the forces to the valve decks and discharge chambers. Such an arrangement is not unusual, and the castings are stiff enough to serve, but it is probably preferable not to put these bending strains through the flange joints. The weight of each tension rod is 530 lb.

Valve Decks, Cages, and Valves.—The valve deck is of cast steel, a dome-shaped plate, 2 in. thick, dished to a 7-ft. radius, and set into a pipe, 60 in. in diameter, 2 in. thick, and 15 in. long. It has seven 13½-in. round openings, and is ribbed around these waterways, to add stiffness and to compensate for the metal removed. Each opening has a knife-edge cross-bar, carrying a 1¾-in. Parson's bronze stud, with thread 2 in. in diameter to hold the valve cage. The valve deck is designed for a maximum unit stress of 2 000 lb. per sq. in.

The valve cage is a steel casting, six-sided, 15 in. in diameter, ⅝ in. thick, carrying three valves on each side and three on the end. The use of steel for the cages caused a little extra cost for machine work, but there was no difficulty in making the castings.

The valve is a screw-valve with a brass seat, outside size 3½-in. pipe thread, with four ribs, and ⅞-in. stem, using 3¾-in. by ⅝-in. rubber valves. The waterway of each valve is approximately 6 sq. in., and the designed pressure of the rubber valve on the seat is 200 lb. per sq. in. The spring is five turns of No. 8 phosphor-bronze wire, arranged for a lift of $\frac{9}{16}$ in., and with a resistance of ½ lb. per sq. in. of valve surface. Valves of two types were used, one with the valve stem cast integral with the seat, and the other with the stem screwed into the seat with a taper thread. The former type has given much the better service.

The weights are:	Valve deck......................	6 300 lb.
	Valve cage	180 "
	Valve seat, stem, etc............	1 "

The valve area per deck is 903 sq. in., giving a water velocity of 3.4 ft. per sec. at nominal rating.

Suction and Discharge Chambers.—The suction or discharge chamber (these are interchangeable) is a globe-shaped T with a top outlet for the air chamber. The body is 60 in. in diameter, and 2½ in. thick, giving a working stress of 1 200 lb. per sq. in. At all openings the thickness is increased the same as at the pump chamber openings; and, at the air chamber outlet, a diaphragm is cast with a 24-in. hole to reinforce the shell.

There is some apparent waste of material in making the suction castings as heavy as the discharge castings, though ordinarily subjected to no pressure; but, in pumps for moderate pressures, such as this one, the possible reduction of weight, while keeping enough metal for rigidity, is not very great. The weight of this casting is 17 000 lb.

Air Chambers.—The air chamber for the suction or discharge end, is 44 in. in diameter, and 2 in. thick, giving a working stress of 1 100 lb. per sq. in. The top of the air chamber is fastened to the steel girder above by bolting through a cast-iron block fitted to the space left, and it helps to stiffen the steel frame. The weight of the air chamber is 7 300 lb.

The relation of the volume of air chambers provided to the capacity and type of pump is of interest. This relation is often a matter of judgment based on experience, rather than of numerical computation, and is affected by questions of structural convenience and patterns available, but a comparison based on a logical theory will indicate to some extent what this experience has determined.

There are two important standards for determining air chambers: one is dependent on the relation of the variable quantity of discharge of the pump as a whole to the flow in the suction or discharge lines, which is usually at practically constant velocity; the second is dependent on the relation of the variation of discharge of a single plunger to the air chambers effectual in absorbing the variable effect of the single plunger, and transmitting a constant effect to the pipe lines, or the suction or discharge chambers.

Considering first the pump as a whole in relation to the line: In the Queen Lane engine, as in any single-acting type, with cranks placed at 120° from each other, assume the pipe-line velocity to be constant:

Let A represent the area of the plunger,
S " " stroke,
N " " revolutions per minute,
a " " angular displacement of the crank,
Q " " the average rate of discharge of the whole pump (3 plungers),
T " " the time, in seconds,
E " " maximum excess of the water quantity delivered by the three plungers compared to the mean,
V " " plunger velocity.

Neglecting the effect of the angularity of the connecting rod, the elementary discharge in the time, dT, is

$$Q\,dT = \frac{3\,A\,S\,N\,dT}{60} \quad \text{......(1)}$$

The discharge of one plunger, which acts alone during the time of excess discharge, is

$$A\,V\,dT = \frac{A\,\pi\,S\,N \sin. a\,dT}{60} \quad \text{......(2)}$$

The elementary variation in volume of the excess water is

$$dE = \frac{A\,\pi\,S\,N}{60} \sin. a\,dT - \frac{3\,A\,S\,N\,dT}{60} \quad \text{......(3)}$$

$$= \frac{A\,S\,N}{60}(\pi \sin. a - 3)\,dT \quad \text{......(4)}$$

and, as the angular velocity is

$$\frac{dA}{dT} = \frac{2\,\pi\,N}{60} \quad \text{......(5)}$$

$$dE = \frac{A\,S}{2}\left(\sin. a - \frac{3}{\pi}\right) da \quad \text{......(6)}$$

To determine the limits between which an excess discharge takes place, put dE equal to 0

$$\frac{AS}{2}\left(\sin. a - \frac{3}{\pi}\right) da = 0 \quad \text{......(7)}$$

$$\sin. a = \frac{3}{\pi} \qquad a = 72^\circ\,44' \text{ and } a = 107^\circ\,16'$$

$$E = \int_{72^\circ\,44'}^{107^\circ\,16'} \frac{A\,S}{2}\left(\sin. a - \frac{3}{\pi}\right) da \quad \text{......(8)}$$

$$E = \frac{A\,S}{2}\left(-\cos. a - \frac{3\,a}{\pi}\right)_{72^\circ\,44'}^{107^\circ\,16'} \quad \text{......(9)}$$

$$= 0.00926\,A\,S \quad \text{......(10)}$$

Call M the volume of the air chambers,

and p the ratio of variation of pressure to the original pressure,

$$p = \frac{0.00926\,A\cdot S}{M} \quad \text{......(11)}$$

For a two-cylinder, double-acting engine, with cranks at 90°,

$$E = 0.042\,A\,S \quad \text{......(12)}$$

A comparison of the engines in Table 2, on this basis, is given in Table 3.

TABLE 3.—COMPARISON OF E, M, AND p.

Reference No.	Make and size.	E	M	p
4	Holly—20 000 000 gal. per day..........	521	618 000	0.00085
5	Allis—6 000 000 " " "	425	100 000	0.0042
6	Snow—7 500 000 " " "	365	118 000	0.0031
7	Bethlehem—15 000 000 gal. per day......	715	304 000	0.0024
8	Philadelphia—20 000 000 gal. per day....	468	394 000	0.0012

Based on the second standard of comparison, the relation of the discharge of a single plunger to the air chambers effectual in absorbing the variable effect of the plunger discharge, in engines such as exemplified in Table 3, an air chamber is placed over each valve deck, all connected by the equalizing air pipes, so that the whole volume of all the air chambers acts to smooth the intermittent action of each single plunger, the variation in volume of air owing to other plungers being negligible.

The variable effect to be absorbed by the air chambers is directly proportional to the quantity of water discharged by that plunger, and to the maximum or average velocity of the plunger (these having a constant relation); using K to express the relative effectiveness of the air chambers, while V represents the mean plunger velocity.

$$K = \frac{M}{A\ V}$$

Table 4 is a comparison of the engines in Table 3 on this basis.

TABLE 4.—COMPARISON OF V AND K.

Reference No.	Make and size.	V	K
4	Holly—20 000 000 gal. per day...........	3.67	196
5	Allis—6 000 000 gal. per day..............	2.92	139
6	Snow—7 500 000 gal. per day..............	4.67	122
7	Bethlehem—15 000 000 gal. per day......	5.33	161
8	Philadelphia—20 000 000 gal. per day....	3.30	124

The agreement shown by the factor, K, in four of these engines is reasonably satisfactory; in Engine No. 4 the size of the air chambers is largely determined by structural considerations, and is probably not an intentional feature of the design.

Plunger.—The plunger is a closed-end pipe, 34½ in. in diameter and 1½ in. thick; the weight is 4 300 lb. The closed end is almost flat, being dished only enough for rigidity, as it is believed that the pointed end is of little value for a plunger running at this slow speed.

Repair Work on the Steam End.—The entire steam end was rebuilt, using the old parts merely as raw material, but re-machining and refitting all parts. The greater part of this work was only the routine of the machine shop, but a few points of interest are to be noted.

Steam Bed-Plates.—The old steam bed-plates carrying the main bearings were retained. In addition to being bolted to the steel frame, each bed-plate was fastened to the cast-iron girder at the tops of the bracing columns by six 2½-in. bolts at each main bearing. The engines had originally but two such bolts, but their length, about 8 ft., and the insufficient stiffness of the bed-plate, permitted the bearings to lift, and required the addition of the four extra bolts. In the light of present experience, the steam bed-plates and the structural steel frames might have been abandoned and new bed-plates provided at no heavy increase in cost.

Main Shaft.—The drag-crank connection, in this type of crank shaft, is the sore thumb of the modern pumping engine. In this old machine, a ball joint with wedge adjusting plates had been provided originally. It was enlarged from 7 to 9 in. (for the 18-in. shaft), and made of high-carbon steel, with hard bronze wearing plates.

The writer is familiar with some of the drag-crank constructions provided in the best modern pumping engines, and considers them invariably insufficient. They are usually designed on the assumption that as there is little or no movement, the governing factor—the only important factor—is structural strength. It is the same mistake (but much more serious) as that made in designing pins for a pin-connected truss. A drag-crank connection should be built on the same lines as a bearing, with moderate unit pressures, and, particularly, with just as efficient a provision for lubrication. The attempt to provide a universal motion in such a style gives a cumbersome and expensive construction, but nothing less will serve.

Main Bearings.—In engines of this type, there seems to be a strong tendency of the shaft and the crank-pin forces to lift the main bearing caps, presumably by stretching the bolts. In these rebuilt engines, the cap is held down by four 3-in., and two 2¾-in. bolts, of 80 000-lb. steel; there is a noticeable amount of lift at the beginning of the upward stroke, more marked at the high-pressure end. The maximum steam cylinder force, in round numbers, is 100 000 lb., giving a bolt stress of 3 200 lb. per sq. in. Bolts have broken without showing any internal defects. This corresponds to a similar condition observed by the writer in some recently built engines of the same size and type, of first-class construction, where four 3½-in. bolts are used for the main bearing cap. Here, too, in some cases, the main cap will lift. This condition will obtain at times when the most careful measurements indicate with reasonable certainty that there is no misalignment of the bearings. It would be expected that bolts stressed to about 3 000 lb. would hold the cap with rigidity and safety. The writer would welcome any light which other members could throw on this subject.

Receivers.—In this reconstruction, the reheating tubes in the steam receivers were removed, and the receiving tanks were used merely as equalizers and separators. It was anticipated that, under the most favorable conditions, the possible gain in reheating would be trifling, and that in service a loss would be more likely than a gain.

Subsequent tests made at the Lardner's Point Pumping Station, in Philadelphia, with a number of identical Holly engines, have confirmed the correctness of this view. At that station, the best record of six engines, amounting to a duty of more than 182 000 000 ft-lb. per 1 000 lb. of wet steam, was made with the reheating coils cut entirely out of service.

A-*Frames.*—The erection of the A-frames developed the fact that it was necessary to converge the two frames of the steam cylinder at the steam end to allow for the expansion of the cylinder on admitting the steam. The final working lines of the hot engine then showed the guides to be parallel.

Exhaust Line.—The exhaust line between the low-pressure cylinder and the condenser was provided with an expansion joint just above the condenser, the line hanging from the cylinder. This is not usually considered necessary, but it is a precaution which is worth while.

Air Pump.—The first of the three engines was fitted with a direct-acting, steam-operated, air pump, as shown in Plate XI; as the exhaust steam ran to the heater line, it was efficiently utilized, and the engine was simplified. The air pump was a standard commercial pump, built of poor material, and of flimsy design, and by breakages caused more trouble than all the rest of the engines. The second and third engines were fitted with air pumps driven directly from an extension arm from the low-pressure plunger cross-head.

The execution of the work, on the basis of unit price contracts, has been simple and convenient, and the completed engines operate in a satisfactory manner. Such weak points as have developed have occurred in the steam end at places where structural limitations prevented any material change of design.

The Director of Public Works, Philadelphia, was Mr. George R. Stearns; the Chief of the Bureau of Water was Fred. C. Dunlap, M. Am. Soc. C. E.; and the work was in charge of the writer. The erection was done by the regular mechanical force of the Bureau, successively under Mr. James Barbour, and Mr. Harry Mellen, the plans having been drawn by Mr. W. E. Kuen. The contractors were the I. P. Morris Company, and the Southwark Foundry and Machine Company, both of Philadelphia.

Acknowledgment is due to Mr. Dunlap for permission to use the data contained in this paper.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1228

FINAL REPORT OF THE SPECIAL COMMITTEE ON UNIFORM TESTS OF CEMENT.*

The President and Members, American Society of Civil Engineers.

Gentlemen:—At the Annual Meeting, held January 18th, 1911, your Committee on Uniform Tests of Cement submitted a final report, as required by resolution passed at the preceding Annual Meeting, and stated that it seemed possible, by conference with a Board of Engineers to be appointed by the Chief of Engineers of the United States Army, to agree upon methods approved by both the Board and your Committee, which it was believed would result in uniform practice by all engineers in the United States; the Society thereupon continued the Committee for one year.

The Army Board was duly appointed, its membership including one of the members of your Committee. Conferences were held or hearings given as follows:

On September 12th, 1911, a hearing was given by the Army Board in New York, attended by representatives of your Committee, by a representative of one of the commercial testing laboratories, by a representative from the United States Bureau of Standards, and by several manufacturers. It appeared at this hearing that a tentative specification for methods of testing had been prepared by representatives of several bureaus in Washington, adopting the methods recommended in previous reports of your Committee, except in regard to the determination of normal consistency and time of setting of cement pastes. Upon the request of your Committee two additional conferences were held, one on November 27th, 1911, the other on January 8th, 1912, with the hope on the part of your Committee of reaching entire agreement, but without favorable result.

* Presented to the Annual Meeting, January 17th, 1912.

In submitting this, its final report, your Committee desires to describe, in some detail, the differences between the proposed methods of making these tests, and to state on these points the reasons for its final recommendations, and to refer to the current practice in this and other countries.

The Vicat apparatus, which is recommended by your Committee for the determination of consistency and time of setting, was originally devised by Vicat about 1818, to ascertain the relative rates of induration of mortars, and although it has since been slightly modified to make it more suitable for determining the time of setting of plastic mortars of neat cement, the principle of the apparatus, the vertical guiding of a weighted wire, remains unchanged. The ball method for determining normal consistency, which has been adopted by the Army Board, is not new, but was used in France before the adoption of the Vicat apparatus for this purpose. The relative merits of the two methods were investigated, with many comparative experiments, by a Commission on Methods of Testing Materials of Construction appointed by the Government of France in 1891. As a result of this investigation, the Commission in 1893 adopted the Vicat apparatus for determining normal consistency; it has since been adopted by the International Association for Testing Materials, and in many countries, as will be shown further on in this report.

The so-called Gillmore wires appear to have been first proposed by M. Antoine Racourt, to whom Gen. Jos. G. Totten, Hon. M. Am. Soc. C. E., refers in his translation of "Essays on Hydraulic and Common Mortars, etc.," by Treussart and others, published in 1842. Gen. Q. A. Gillmore, M. Am. Soc. C. E., in his "Practical Treatise on Limes, Hydraulic Cements, and Mortars" refers to these wires as having been used by Gen. Totten prior to 1830, and recommends their use for determining time of setting; it does not appear that they have ever been used for determining normal consistency, for which purpose they are not suitable.

When your Committee began the duty assigned to it, it took into consideration the ball method and the Vicat apparatus for testing consistency. A great many tests were made by the members of the Committee to determine the relative value of these methods, after which the method by use of the Vicat apparatus was formulated, and the Committee proceeded to test it in comparison with other methods in common use. The tests were arranged to include a comparison of the method of mixing pastes and mortars and moulding test pieces recommended by the Committee with other methods. Accordingly a meeting was held at the laboratory of the Atlantic Avenue Improvement of the Long Island Railroad, under the direction of your Committee, and in the presence of several of its members, at which were present representatives of seven laboratories of recognized standing. The cement was prepared carefully by mixing with a garden rake on a clean papered floor; then sifting in long thin layers one on top of the other and again mixing with the rake and sifting into a barrel from which it was used. The operators were all experienced in testing cement, and, with a single exception, were accustomed to daily practice; those who took part assembled in an outer room, from which each in turn entered the laboratory where he made determinations for consistency and also made a set of 20 briquettes, all in accordance with the Committee's methods, and at the same time made a set of ten briquettes in accordance with his own method. For the purpose of uniformity, the weighing of the cement and measuring of the water was done by one person while the manipulation of the Vicat apparatus was entrusted to another. After completion of his work each operator remained in the laboratory, affording no opportunity for exchange of views with those who had not performed the experiments. The briquettes were all kept under the same conditions, stored in moist air for 24 hours, and then immersed in water and kept at a temperature as near 70° Fahr. as possible, and after a specified period were removed from the water, the excess moisture absorbed by blotting paper, the briquettes weighed and broken. The result of these tests showed that the several operators agreed as to the proper percentage of water required for normal consistency determined by the Vicat apparatus. In making briquettes by their own methods the operators used different consistencies, from wet paste to material so dry that it required pounding in the mould with a mallet, the percentage of water varying from 16 to 24%; more consistent results were obtained with the Committee's consistency and by the methods recommended by the Committee than with the methods of the operators.

Arrangements were then made for another series of experiments with five samples of Portland cement and four samples of natural cement, carefully prepared as before described, hermetically sealed in tin cans and sent to some 26 testing laboratories in various part of the country with a request to test the cements on a given date in accordance with the methods formulated by your Committee, and to report the results to the Secretary on certain dates. These results were collated and a study showed such agreement in regard to consistency, strength, and other tests as to satisfy the Committee that by its methods concordant results could be secured by different operators in different parts of the country. In order to compare the ball test for normal consistency with the Vicat apparatus method, samples of cement were prepared and sent out to several members of the Committee; simultaneous tests of the two methods were made, and the results were conclusive, the Vicat apparatus giving more concordant results than the ball method. The Committee does not wish it to be inferred that any method yet proposed for determining consistency will always prove exact, for no such method has been devised, but it does fully believe that, by the method recommended, operators in different parts of the country can secure more concordant results than can be obtained by any other method yet proposed.

The Army Board has adopted the ball test, which was recently defined as follows:

"A quantity of cement paste should be mixed in the manner hereinafter described under Tensile Tests and quickly formed into a ball about 2 in. in diameter. The ball should then be dropped upon a hard, smooth surface from a height of 2 ft. The paste is of normal consistency when the ball does not crack and does not flatten more than one-half of its original diameter."

The ball test in some form has been in use for many years as a rough and ready means of judging the consistency of mortar. Quite recently a number of experiments with the test were made under the direction of the Committee by experts in testing cement, with the result that variations in the percentage of water amounting to 2 or 3% of the weight of the cement, or about 10 to 15% of the weight of the water, might not be detected by this test of consistency.

The method of forming the ball can hardly be defined so that the work put on the paste by different operators in shaping it will be the same; if the ball is oblong, rather than spherical, the amount of flattening will depend considerably on whether the ball is dropped with the longer or shorter axis vertical. The specification above quoted may be made much more definite in this respect, and the amount of flattening can be better defined; such changes may have been made in the more recent revisions by the Army Board, but even with this assumed your Committee is convinced that the test it recommends is the better; it requires less time for the preparation of the sample of paste, but, on the other hand, the application of the cylinder requires more time than dropping the ball, and the complete test with the Vicat apparatus may require a little more time than the ball test. The difference, however, is not important, since either test is quickly made, and the cost is trifling.

The percentage of water adopted by the Army Board for mortars containing one part of cement to three parts of Ottawa sand is uniformly 1% greater than recommended by your Committee. Additional experiments made recently by your Committee, confirm its previous recommendation.

In the tests for time of setting it is sought to determine two stages in the process, one when the paste ceases to be plastic, termed the "initial set," the other when it will support a given weight on a given area, termed the "final set." Neither term is absolutely correct, particularly the term "final set"; each as used depends largely on the instrument for making the test, but with this stated and its application carefully described and followed the terms become readily understood.

Your Committee recommends the use of the Vicat needle for determining these stages, while the Board of Army Engineers adopts the Gillmore needles. It is believed that the phrase, "Vicat needle apparatus" has given the incorrect impression of complexity. The apparatus consists simply of a single rod of given weight and given diameters at the ends, moving vertically in a guide; in its use, the end of the rod is brought into contact with the paste and held lightly by a thumb-screw, then released with a minimum of vibration or jarring. The Gillmore needles are wires of given diameters carrying given weights; two are required for determination of time of setting; they are applied by hand, without guides, and the results depend much on the steadiness of hand and the skill of the operator. It seems to your Committee that there should be no doubt that the Vicat needle is the better instrument; although slightly more expensive, it does not increase the cost of a laboratory equipment more than 3 or 4 per cent.

Of the two stages in the process of setting, the initial set is of the greater importance, since it marks the moment when the setting becomes appreciable, and it is generally believed that if the paste is broken up after this stage is reached its final strength will be reduced. In the method recommended by your Committee the sample for testing is formed from the paste with a minimum of manipulation; care is taken not to compress the paste, and the surface to which the needle is to be applied is formed by slicing off the paste above a given thickness of sample without pressure upon the sample, the condition of the paste at the surface being identical with that in the mass. The thickness of the mass is a little more than 1½ in. and is a definite quantity. When the sample is first formed, the Vicat needle penetrates readily through the entire thickness or depth of the mass; as the setting proceeds, a moment arrives when the needle no longer penetrates entirely through, but stops when within a short, specified distance from the lower surface. This is taken as the initial set. For such a test it is obvious that the movement of the needle must be guided, for unsteadiness in its lateral support would have a great influence on the amount of penetration. For the application of the Gillmore needle, a thin pat (about ½ in. thick) is formed on a glass plate by troweling. The amount of troweling does not admit of clear definition, and will vary widely with different operators. It is a matter of common observation that a troweled surface differs much in density from the mass, and in the pats for the Gillmore needles this difference will be highly variable because of the difference in troweling. This is important, since the initial set is determined with the Gillmore needle not by penetration of the mass, but by a slight indentation of its surface; thus depending, not only on the chemical action of setting, but on the variable physical preparation of the surface, as well as on the variable condition of the atmosphere, which will affect a surface more than a mass.

It has been stated that the Gillmore needle test requires less time. There can be no question that the Vicat sample for testing is more quickly formed than the Gillmore pat, while a single application of the Gillmore needle will require less time than the Vicat needle; if a single test were made at the specified limit of time, to determine simply whether the initial set had occurred, the Vicat test would be the quicker; if repeated applications of the needle were made to ascertain at what moment the set occurred and the number of applications were large, the Vicat method might require more time. The difference would be small if the Gillmore needle were used with great care, and would not be important in any case.

In the judgment of your Committee the determination of initial set is of much importance, and is much better done with the Vicat needle used in the manner it recommends; in the determination of normal consistency, however, the superiority of the Vicat method over the ball method, while appreciable, is less marked.

The determination of final set is of less importance than the determination of initial set; in both methods the test is of indentation, not penetration, the difference being mainly in the nature of the surface tested. For reasons already given, the surface of the sample used with the Vicat needle represents the mass more fairly than the troweled surface of the pat used with the Gillmore needle.

Your Committee in its endeavor to reach an agreement with the Army Board offered to accept the less desirable ball method for determining normal consistency if the Board would adopt the Vicat needle for time of setting. By the rejection of this offer, your Committee was brought to the question whether, for the sake of complete agreement with the Army Board, substantial agreement having already been reached, it would recommend for the test of time of setting a method which it believed to be greatly inferior in regard to an important test, constituting a decided retrogression in methods for testing cement.* The methods of the Army Board have the concurrence of a departmental committee representing several branches of the United States Government, and your Committee, having asked at the last Annual Meeting for an extension of time in order to secure uniformity in methods, felt strongly the desirability of effecting entire agreement, and has given the questions of difference renewed and most earnest consideration.

* One member of the Committee has expressed dissent from this statement of the case, and believes that from a practical point of view the results obtained by the ball method for determining normal consistency and the Gillmore needles for time of setting are as satisfactory as those given by the Vicat apparatus.

The Vicat apparatus recommended by your Committee in its first preliminary report in 1903, had been in use for many years in many laboratories and had been thoroughly tried out in the laboratory of the City of Philadelphia. Since 1903 its use has been greatly extended. Previous to the last Annual Meeting, the Secretary of this Committee addressed a letter of inquiry to testing laboratories in the United States, and received replies from 143; of these, 93 reported the use of the methods recommended by your Committee, and 72 reported them very satisfactory; 12 were from Army Engineers who used the methods prescribed by the Engineer Corps in Professional Paper 28; 30 used their own methods, and 8 reported that they did not make cement tests; of the total number of replies, 19 used the Gillmore needles and 114 used the Vicat apparatus; 2 used their own methods, and the remainder, as previously stated, did not make cement tests.

The method recommended by your Committee being thus supported in this country, the practice in foreign countries was investigated, with the following results:

SUMMARY OF METHODS SPECIFIED FOR DETERMINING TIME OF SETTING AND NORMAL CONSISTENCY IN FOREIGN COUNTRIES.

Country.		Time of Setting.	Normal Consistency.
Belgium	(*a*)	Vicat Needle.	Vicat Apparatus.
Denmark	(*b*)	" "	" "
France	(*a*)	" "	" "
Holland	(*c*)	" "	" "
Hungary	(*d*)	" "	" "
Italy	(*a*)	" "	" "
International Assn. Test. Materials.		" "	" "
Russia	(*e*)	" "	" "
Austria	(*d*)	" "	Boehme Hammer Apparatus.
Germany	(*a*)	" "	" " "
Switzerland	(*f*)	" "	" " "
England	(*g*)	" "	Note 1.
Canada	(*h*)	Note 2.	Note 3.

(*a*) Ministry of Public Works.
(*b*) Danish States Testing Laboratory.
(*c*) Royal Institute of Engineers.
(*d*) Association of Engineers and Architects.
(*e*) Ministry of Ways and Communications.
(*f*) Ministerial Regulations.
(*g*) Engineering Standards Committee.
(*h*) Canadian Society of Civil Engineers.

Note 1.—The cement shall be mixed with such proportion of water that the mixture shall be plastic when filled into the Vicat mould. The gauging shall be completed before the signs of setting occur.

Note 2.—The cement shall be considered as having taken "initial set" when a wire $\frac{1}{12}$ in. in diameter, loaded to weigh $\frac{1}{4}$ lb., shall leave a distinct mark on the pat, but not appreciably penetrate the surface, and the "final" or "hard set" when a wire $\frac{1}{24}$ in. in diameter, loaded to weigh 1 lb., shall leave a distinct mark, but not appreciably penetrate the surface.

Note 3.—For a cement 75% of which will pass a No. 200 sieve, a maximum of 22% of water, and an additional 1% of water for each extra 5% of cement that will pass the No. 200 sieve.

Your Committee would direct attention to the Report of The Engineering Standards Committee of England, supported by:

The Institution of Civil Engineers,
The Institution of Mechanical Engineers,
The Institution of Naval Architects,
The Iron and Steel Institute,
The Institution of Electrical Engineers,

dated August, 1910, containing a revision of the British Standard Specifications for Portland Cement, in which the following statement is to be found concerning the determination of the time of setting:

"Since the issue of the first revision of the Specification the Committee has continued its investigation into the question of the determination of the initial setting time of cement. It was found that while the final setting times determined by the British Standard and Vicat Needles approximated very closely, the initial setting time as determined by the British Standard Needle differed considerably from that given by the Vicat needle which is in general use, and also from that obtained by the rough and ready test of the finger nail.

"It was considered preferable that one instrument only should be specified for determining the initial and final setting times of cement, and the Vicat Needle has been adopted for that purpose."

By this action a modified form of the Gillmore needle was superseded.

At the Sixth International Congress for Testing Materials, held at Copenhagen in 1909, an official report on the progress in methods of testing hydraulic cements was presented by R. Feret, Ingénieur en Chef, Laboratoire d'Essai des Ponts et Chaussées, at Boulogne sur Mer. In this report M. Feret makes the following comments on methods for determining the duration of setting:

"The use of the Vicat needle continues to be the only practical method in use for the determination of the duration of the period of setting of hydraulic cements. The appliance is one of extreme simplicity, but its readings are sometimes uncertain, especially when it is a question of determining the end of the period; besides the readings are of a purely conventional character and do not appear always to bear a sufficiently constant relation to the duration of the setting period of the mortars of actual practice.

"The discovery of more exact methods has therefore been attempted."

Attention is further called to a paper by W. C. Reibling and F. D. Reyes, on "The Setting Properties of Portland Cement," contained in Vol. VI, No. 3, Section A, June, 1911, of the *Philippine Journal of Science,* published by the Bureau of Science of the Philippine Government, in which the authors make the following comments on the Vicat apparatus:

"Throughout our work, several standard methods were employed for determining the time of the initial and final set. The method employing the Vicat needle as adopted by the American Society for Testing Materials was found to be the most consistent with the manner in which the cement is used in actual work. It is reliable, impartial and accurate."

In view of all this evidence, the Committee does not feel justified in modifying its previous recommendation of the Vicat apparatus.

The methods recommended by your Committee imply the use of well-equipped laboratories, such as are now usually found in connection with large works of construction, and it is believed they are described in sufficient detail to enable skilled operators to obtain concordant results without communicating with one another. This is shown by comparing past and present practice in regard to normal consistency. When your Committee began its work, the consistency adopted in the different laboratories had a wide range, from very soft to very dry paste; now the practice is virtually uniform in the United States, and this is due, the Committee firmly believes, to the general use of the methods recommended by it in previous reports, or, in other words, to training with the Vicat apparatus.

Where the construction work is of small extent, field tests of less definite character will be made, depending on the facilities and time available. These are so variable in extent and kind, and can be so readily specified by the Engineer, that it has not been deemed advisable or practicable to enumerate and describe them.

Since its last report your Committee has made several verbal changes in its recommendations. Methods for igniting cement and for determining insoluble residue have been inserted, although it is apprehended that the latter determination may prove to be of little value. The Committee now recommends the clip with roller points, which has been used successfully and by which central breaks may be obtained in most cases. The final recommendations are submitted herewith.

As in former reports, the significance of each test is stated, as well as the method of carrying it out.

For the convenience of engineers who may desire to incorporate in their specifications the methods recommended, a condensed draft is also submitted, in which discussion is omitted.

In accordance with the resolution passed at the last Annual Meeting, the duty of this Committee is concluded with this report.

Respectfully submitted on behalf of the Committee,

GEORGE S. WEBSTER, *Chairman.*
RICHARD L. HUMPHREY, *Secretary.*

JANUARY 17TH, 1912.

METHODS FOR TESTING CEMENT.*

SAMPLING.

1.—*Selection of Sample.*—The selection of samples for testing should be left to the engineer. The number of packages sampled and the quantity taken from each package will depend on the importance of the work and the facilities for making the tests.

2.—The samples should fairly represent the material. When the amount to be tested is small it is recommended that one barrel in ten be sampled; when the amount is large it may be impracticable to take samples from more than one barrel in thirty or fifty. When the samples are taken from bins at the mill one for each fifty to two hundred barrels will suffice.

3.—Samples should be passed through a sieve having twenty meshes per linear inch, in order to break up lumps and remove foreign material; the use of this sieve is also effective to obtain a thorough mixing of the samples when this is desired. To determine the acceptance or rejection of cement it is preferable, when time permits, to test the samples separately. Tests to determine the general characteristics of a cement, extending over a long period, may be made with mixed samples.

4.—*Method of Sampling.*—Cement in barrels should be sampled through a hole made in the head, or in one of the staves midway between the heads, by means of an auger or a sampling iron similar to that used by sugar inspectors; if in bags, the sample should be taken from surface to center; cement in bins should be sampled in such a manner as to represent fairly the contents of the bin. Sampling from bins is not recommended if the method of manufacture is such that ingredients of any kind are added to the cement subsequently.

CHEMICAL ANALYSIS.

5.—*Significance.*—Chemical analysis may serve to detect adulteration of cement with inert material, such as slag or ground limestone, if in considerable amount. It is useful in determining whether certain constituents, such as magnesia and sulphuric anhydride, are present in inadmissible proportions.

6.—The determination of the principal constituents of cement, silica, alumina, iron oxide, and lime, is not conclusive as an indication of quality. Faulty cement results more frequently from imperfect preparation of the raw material or defective burning than from incorrect proportions. Cement made from material ground very finely and thoroughly burned may contain much more lime than the amount usually present, and still be perfectly sound. On the other hand,

* Accompanying Final Report of Special Committee on Uniform Tests of Cement, dated January 17th, 1912.

cements low in lime may, on account of careless preparation of the raw material, be of dangerous character. Furthermore, the composition of the product may be so greatly modified by the ash of the fuel used in burning as to affect in a great degree the significances of the results of analysis.

7.—*Methods.*—The methods to be followed, except for determining the loss on ignition, should be those proposed by the Committee on Uniformity in the Analysis of Materials for the Portland Cement Industry, reported in the *Journal* of the Society for Chemical Industry, Vol. 21, page 12, 1902; and published in *Engineering News,* Vol. 50, p. 60, 1903; and in *Engineering Record,* Vol. 48, p. 49, 1903, and in addition thereto, the following:

(*a*) The insoluble residue may be determined as follows: To a 1-gramme sample of the cement are added 30 cu. cm. of water and 10 cu. cm. of concentrated hydrochloric acid, and then warmed until effervescence ceases, and digested on a steam bath until dissolved. The residue is filtered, washed with hot water, and the filter paper and contents digested on the steam bath in a 5% solution of sodium carbonate. This residue is filtered, washed with hot water, then with hot hydrochloric acid, and finally with hot water, and then ignited at a red heat and weighed. The quantity so obtained is the insoluble residue.

(*b*) The loss on ignition shall be determined in the following manner: ½ gramme of cement is heated in a weighed platinum crucible, with cover, for 5 minutes with a Bunsen burner (starting with a low flame and gradually increasing to its full height) and then heated for 15 minutes with a blast lamp; the difference between the weight after cooling and the original weight is the loss on ignition. The temperature should not exceed 900° cent., or a low red heat; the ignition should preferably be made in a muffle.

Specific Gravity.

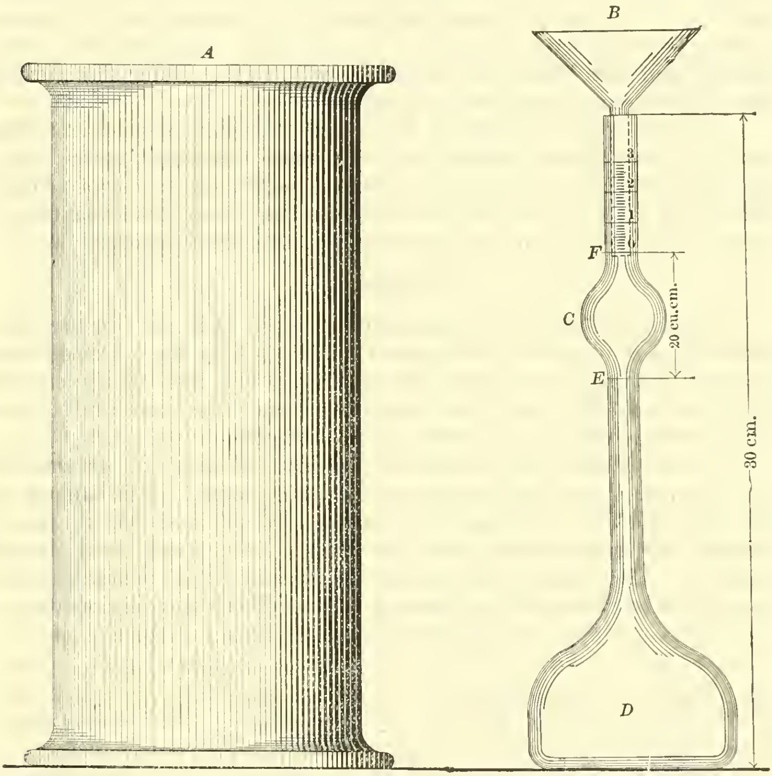
8.—*Significance.*—The specific gravity of cement is lowered by adulteration and hydration, but the adulteration must be considerable to be detected by tests of specific gravity.

9.—Inasmuch as the differences in specific gravity are usually very small, great care must be exercised in making the determination.

10.—*Apparatus.*—The determination of specific gravity should be made with a standardized Le Chatelier apparatus. This consists of a flask (*D*), Fig. 1, of about 120 cu. cm. capacity, the neck of which is about 20 cm. long; in the middle of this neck is a bulb (*C*), above and below which are two marks (*F*) and (*E*); the volume between these two marks is 20 cu. cm. The neck has a diameter of about 9 mm., and is graduated into tenths of cubic centimeters above the mark (*F*).

11.—Benzine (62° Beaumé naphtha) or kerosene free from water should be used in making the determination.

12.—*Method.*—The flask is filled with either of these liquids to the lower mark (*E*), and 64 grammes of cement, cooled to the temperature of the liquid, is slowly introduced through the funnel (*B*), (the stem of which should be long enough to extend into the flask to the top of the bulb (*C*)), taking care that the cement does not adhere to the sides of the flask, and that the funnel does not touch the liquid.



LE CHATELIER S SPECIFIC GRAVITY APPARATUS.

FIG. 1.

After all the cement is introduced, the level of the liquid will rise to some division of the graduated neck; this reading, plus 20 cu. cm., is the volume displaced by 64 grammes of the cement.

13.—The specific gravity is then obtained from the formula

$$\text{Specific gravity} = \frac{\text{Weight of cement, in grammes,}}{\text{Displaced volume, in cubic centimeters.}}$$

14.—The flask, during the operation, is kept immersed in water in a jar (A), in order to avoid variations in the temperature of the liquid in the flask, which should not exceed ½° cent. The results of repeated tests should agree within 0.01. The determination of specific gravity should be made on the cement as received; if it should fall below 3.10, a second determination should be made after igniting the sample in a covered dish, preferably of platinum, at a low red heat not exceeding 900° cent. The sample should be heated for 5 minutes with a Bunsen burner (starting with a low flame and gradually increasing to its full height) and then heated for 15 minutes with a blast lamp; the ignition should preferably be made in a muffle.

15.—The apparatus may be cleaned in the following manner: The flask is inverted and shaken vertically until the liquid flows freely, and then held in a vertical position until empty; any traces of cement remaining can be removed by pouring into the flask a small quantity of clean liquid benzine or kerosene and repeating the operation.

Fineness.

16.—*Significance.*—It is generally accepted that the coarser particles in cement are practically inert, and it is only the extremely fine powder that possesses cementing qualities. The more finely cement is pulverized, other conditions being the same, the more sand it will carry and produce a mortar of a given strength.

17.—*Apparatus.*—The fineness of a sample of cement is determined by weighing the residue retained on certain sieves. Those known as No. 100 and No. 200, having approximately 100 and 200 wires per linear inch, respectively, should be used. They should be 8 in. in diameter. The frame should be of brass, 8 in. in diameter, and the sieve of brass wire cloth conforming to the following requirements:

No. of sieve.	Diameter of wire.	Meshes per Linear Inch.	
		Warp.	Woof.
100	0.0042 to 0.0048 in.	95 to 101	93 to 103
200	0.0021 to 0.0023 "	192 to 203	190 to 205

The meshes in any smaller space, down to 0.25 in., should be proportional in number.

18.—*Method.*—The test should be made with 50 grammes of cement, dried at a temperature of 100° cent. (212° Fahr.).

19.—The cement is placed on the No. 200 sieve, which, with pan and cover attached, is held in one hand in a slightly inclined position, and moved forward and backward about 200 times per minute, at the same time striking the side gently, on the up stroke, against the palm of the other hand. The operation is continued until not more than 0.05 gramme will pass through in one minute. The residue is weighed, then placed on the No. 100 sieve, and the operation repeated. The work may be expedited by placing in the sieve a few large steel shot, which should be removed before the final one minute of sieving. The sieves should be thoroughly dry and clean.

Normal Consistency.

20.—*Significance.*—The use of a proper percentage of water in making pastes* and mortars for the various tests is exceedingly important and affects vitally the results obtained.

21.—The amount of water, expressed in percentage by weight of the dry cement, required to produce a paste of plasticity desired, termed "normal consistency," should be determined with the Vicat apparatus in the following manner:

22.—*Apparatus.*—This consists of a frame (*A*), Fig. 2, bearing a movable rod (*B*), weighing 300 grammes, one end (*C*) being 1 cm. in diameter for a distance of 6 cm., the other having a removable needle (*D*), 1 mm. in diameter, 6 cm. long. The rod is reversible, and can be held in any desired position by a screw (*E*), and has midway between the ends a mark (*F*) which moves under a scale (graduated to millimeters) attached to the frame (*A*). The paste is held in a conical, hard-rubber ring (*G*), 7 cm. in diameter at the base, 4 cm. high, resting on a glass plate (*H*) about 10 cm. square.

23.—*Method.*—In making the determination, the same quantity of cement as will be used subsequently for each batch in making the test pieces, but not less than 500 grammes, with a measured quantity of water, is kneaded into a paste, as described in Paragraph 45, and quickly formed into a ball with the hands, completing the operation by tossing it six times from one hand to the other, maintained about 6 in. apart; the ball resting in the palm of one hand is pressed into the larger end of the rubber ring held in the other hand, completely filling the ring with paste; the excess at the larger end is then removed by a single movement of the palm of the hand; the ring is then placed on its larger end on a glass plate and the excess paste at the smaller end is sliced off at the top of the ring by a single oblique stroke of a trowel held at a slight angle with the top of the ring. During these operations care must be taken not to compress the paste. The paste confined in the ring, resting on the plate, is placed under the rod, the larger end of which is brought in contact with the surface of the paste; the scale is then read, and the rod quickly released.

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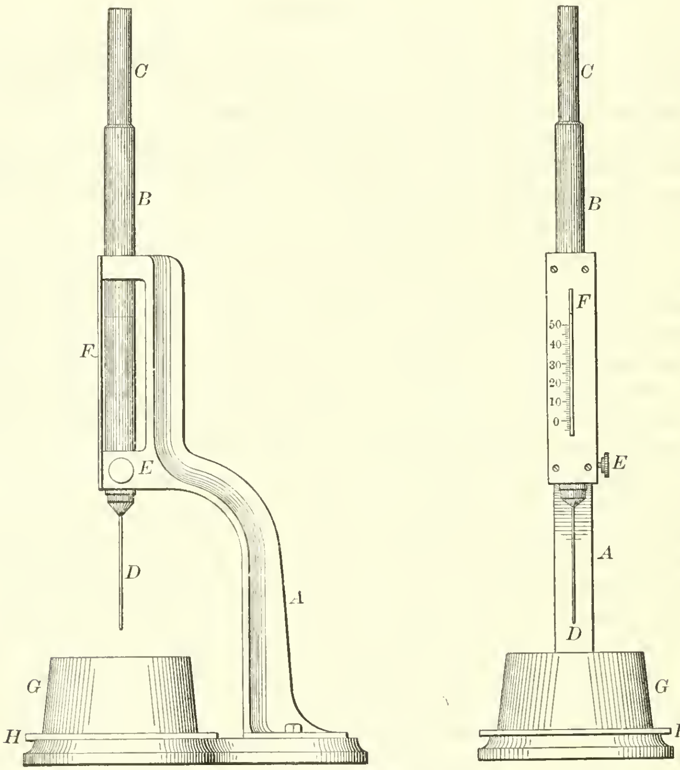
*The term "paste" is used in this report to designate a mixture of cement and water, and the word "mortar" to designate a mixture of cement, sand, and water.

24.—The paste is of normal consistency when the cylinder settles to a point 10 mm. below the original surface in one-half minute after being released. The apparatus must be free from all vibrations during the test.

25.—Trial pastes are made with varying percentages of water until the normal consistency is obtained.



- VICAT APPARATUS

Fig. 2.

26.—Having determined the percentage of water required to produce a paste of normal consistency, the percentage required for a mortar containing, by weight, one part of cement to three parts of standard Ottawa sand, is obtained from the following table, the amount being a percentage of the combined weight of the cement and sand.

PERCENTAGE OF WATER FOR STANDARD MORTARS.

Neat.	One cement, three standard Ottawa sand.	Neat.	One cement, three standard Ottawa sand.	Neat.	One cement, three standard Ottawa sand.
15	8.0	23	9.3	31	10.7
16	8.2	24	9.5	32	10.8
17	8.3	25	9.7	33	11.0
18	8.5	26	9.8	34	11.2
19	8.7	27	10.0	35	11.3
20	8.8	28	10.2	36	11.5
21	9.0	29	10.3	37	11.7
22	9.2	30	10.5	38	11.8

TIME OF SETTING.

27.—*Significance.*—The object of this test is to determine the time which elapses from the moment water is added until the paste ceases to be plastic (called the "initial set"), and also the time until it acquires a certain degree of hardness (called the "final set" or "hard set"). The former is the more important, since, with the commencement of setting, the process of crystallization begins. As a disturbance of this process may produce a loss of strength, it is desirable to complete the operation of mixing or moulding or incorporating the mortar into the work before the cement begins to set.

28.—*Apparatus.*—The initial and final set should be determined with the Vicat apparatus described in Paragraph 22.

29.—*Method.*—A paste of normal consistency is moulded in the hard-rubber ring, as described in Paragraph 23, and placed under the rod (*B*), the smaller end of which is then carefully brought in contact with the surface of the paste, and the rod quickly released.

30.—The initial set is said to have occurred when the needle ceases to pass a point 5 mm. above the glass plate; and the final set, when the needle does not sink visibly into the paste.

31.—The test pieces should be kept in moist air during the test; this may be accomplished by placing them on a rack over water contained in a pan and covered by a damp cloth; the cloth to be kept from contact with them by means of a wire screen; or they may be stored in a moist box or closet.

32.—Care should be taken to keep the needle clean, as the collection of cement on the sides of the needle retards the penetration, while cement on the point may increase the penetration.

33.—The time of setting is affected not only by the percentage and temperature of the water used and the amount of kneading the paste receives, but by the temperature and humidity of the air, and its determination is, therefore, only approximate.

Standard Sand.

34.—The sand to be used should be natural sand from Ottawa, Ill., screened to pass a No. 20 sieve, and retained on a No. 30 sieve. The sieves should be at least 8 in. in diameter; the wire cloth should be of brass wire and should conform to the following requirements:

No. of sieve.	Diameter of wire.	Meshes per Linear Inch.	
		Warp.	Woof.
20	0.016 to 0.017 in.	19.5 to 20.5	19 to 21
30	0.011 to 0.012 "	29.5 to 30.5	28.5 to 31.5

Sand which has passed the No. 20 sieve is standard when not more than 5 grammes passes the No. 30 sieve in one minute of continuous sifting of a 500-gramme sample.*

Form of Test Pieces.

35.—For tensile tests the form of test piece shown in Fig. 3 should be used.

36.—For compressive tests, 2-in. cubes should be used.

Moulds.

37.—The moulds should be of brass, bronze, or other non-corrodible material, and should have sufficient metal in the sides to prevent spreading during moulding.

38.—Moulds may be either single or gang moulds. The latter are preferred by many. If used, the types shown in Figs. 4 and 5 are recommended.

39.—The moulds should be wiped with an oily cloth before using.

Mixing.

40.—The proportions of sand and cement should be stated by weight; the quantity of water should be stated as a percentage by weight of the dry material.

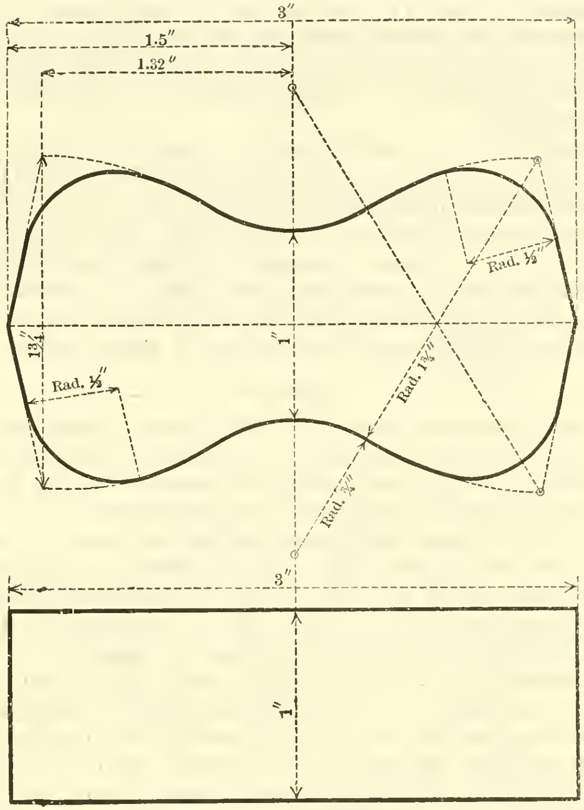
41.—The metric system is recommended because of the convenient relation of the gramme and the cubic centimeter.

42.—The temperature of the room and of the mixing water should be maintained as nearly as practicable at 21° cent. (70° Fahr.).

43.—The quantity of material to be mixed at one time depends on the number of test pieces to be made; 1 000 grammes is a convenient quantity to mix by hand methods.

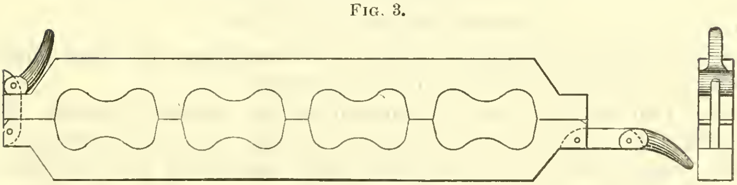
44.—The Committee has investigated the various mechanical mixing machines thus far devised, but cannot recommend any of them, for

* This sand may now (1912) be obtained from the Ottawa Silica Co., at a cost of two cents per pound, f. o. b. cars, Ottawa, Ill.



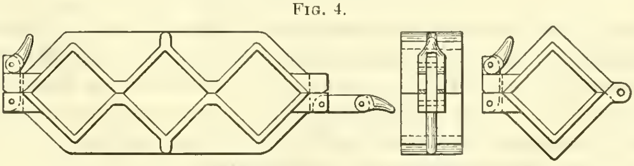
DETAILS FOR BRIQUETTE.

Fig. 3.



DETAILS FOR GANG MOULD.

Fig. 4.



MOULD FOR COMPRESSION TEST PIECES

Fig. 5.

the following reasons: (1) the tendency of most cement is to "ball up" in the machine, thereby preventing working it into a homogeneous paste; (2) there are no means of ascertaining when the mixing is complete without stopping the machine; and (3) it is difficult to keep the machine clean.

45.—*Method.*—The material is weighed, placed on a non-absorbent surface (preferably plate glass), thoroughly mixed dry if sand be used, and a crater formed in the center, into which the proper percentage of clean water is poured; the material on the outer edge is turned into the center by the aid of a trowel. As soon as the water has been absorbed, which should not require more than one minute, the operation is completed by vigorously kneading with the hands for one minute. During the operation the hands should be protected by rubber gloves.

Moulding.

46.—The Committee has not been able to secure satisfactory results with existing moulding machines; the operation of machine moulding is very slow; and is not practicable with pastes or mortars containing as large percentages of water as herein recommended.

47.—*Method.*—Immediately after mixing, the paste or mortar is placed in the moulds with the hands, pressed in firmly with the fingers, and smoothed off with a trowel without ramming. The material should be heaped above the mould, and, in smoothing off, the trowel should be drawn over the mould in such a manner as to exert a moderate pressure on the material. The mould should then be turned over and the operation of heaping and smoothing off repeated.

48.—A check on the uniformity of mixing and moulding may be afforded by weighing the test pieces on removal from the moist closet; test pieces from any sample which vary in weight more than 3% from the average should not be considered.

Storage of the Test Pieces.

49.—During the first 24 hours after moulding, the test pieces should be kept in moist air to prevent drying.

50.—Two methods are in common use to prevent drying: (1) covering the test pieces with a damp cloth, and (2) placing them in a moist closet. The use of the damp cloth, as usually carried out, is objectionable, because the cloth may dry out unequally and in consequence the test pieces will not all be subjected to the same degree of moisture. This defect may be remedied to some extent by immersing the edges of the cloth in water; contact between the cloth and the test pieces should be prevented by means of a wire screen, or some similar arrangement. A moist closet is so much more effective in securing uniformly moist air, and is so easily devised and so inexpensive, that the use of the damp cloth should be abandoned.

51.—A moist closet consists of a soapstone or slate box, or a wooden box lined with metal, the interior surface being covered with felt or broad wicking kept wet, the bottom of the box being kept covered with water. The interior of the box is provided with glass shelves on which to place the test pieces, the shelves being so arranged that they may be withdrawn readily.

52.—After 24 hours in moist air, the pieces to be tested after longer periods should be immersed in water in storage tanks or pans made of non-corrodible material.

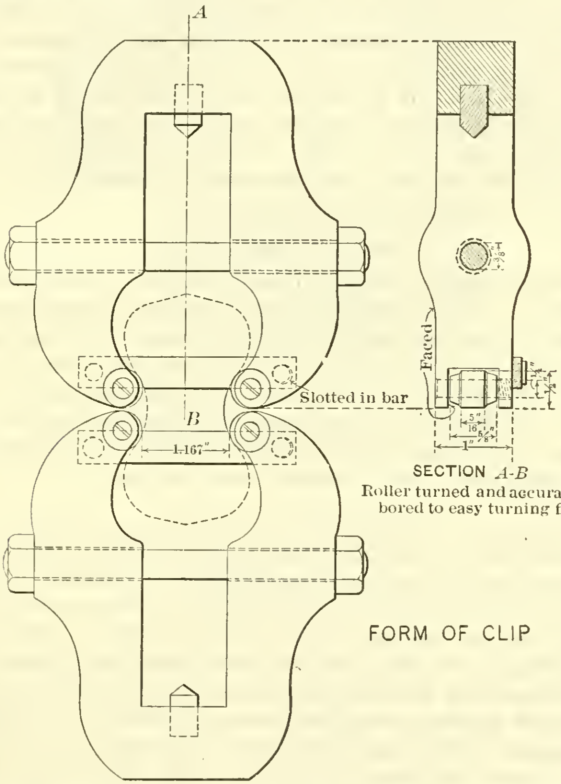


Fig. 6.

53.—The air and water in the moist closet and the water in the storage tanks should be maintained as nearly as practicable at 21° cent. (70° Fahr.).

Tensile Strength.

54.—The tests may be made with any standard machine.

55.—The clip is shown in Fig. 6. It must be made accurately, the pins and rollers turned, and the rollers bored slightly larger than the pins so as to turn easily. There should be a slight clearance at each end of the roller, and the pins should be kept properly lubricated and free from grit. The clips should be used without cushioning at the points of contact.

56.—Test pieces should be broken as soon as they are removed from the water. Care should be observed in centering the test pieces in the testing machine, as cross strains, produced by imperfect centering, tend to lower the breaking strength. The load should not be applied too suddenly, as it may produce vibration, the shock from which often causes the test piece to break before the ultimate strength is reached. The bearing surfaces of the clips and test pieces must be kept free from grains of sand or dirt, which would prevent a good bearing. The load should be applied at the rate of 600 lb. per min. The average of the results of the test pieces from each sample should be taken as the test of the sample. Test pieces which do not break within ¼ in. of the center, or are otherwise manifestly faulty, should be excluded in determining average results.

Compressive Strength.

57.—The tests may be made with any machine provided with means for so applying the load that the line of pressure is along the axis of the test piece. A ball-bearing block for this purpose is shown in Fig. 7. Some appliance should be provided to facilitate placing the axis of the test piece exactly in line with the center of the ball-bearing.

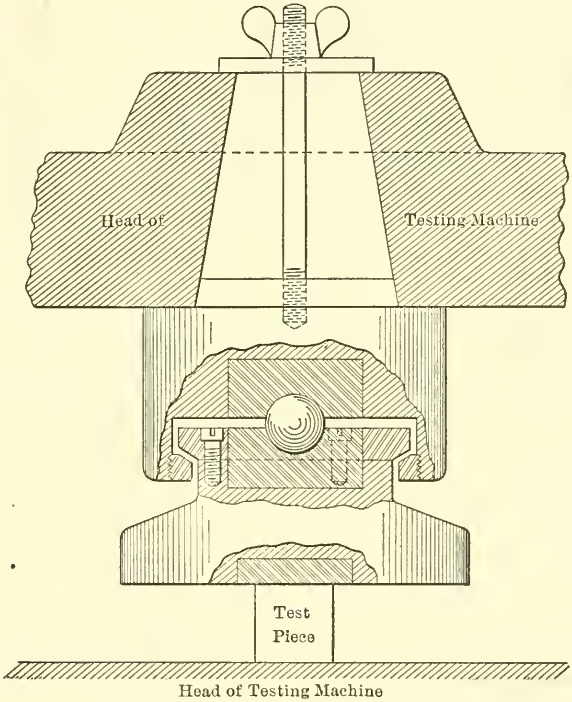
58.—The test piece should be placed in the testing machine, with a piece of heavy blotting paper on each of the crushing faces, which should be those that were in contact with the mould.

Constancy of Volume.

59.—*Significance.*—The object is to detect those qualities which tend to destroy the strength and durability of a cement. Under normal conditions these defects will in some cases develop quickly, and in other cases may not develop for a considerable time. Since the detection of these destructive qualities before using the cement in construction is essential, tests are made not only under normal conditions but under artificial conditions created to hasten the development of these defects. Tests may, therefore, be divided into two classes: (1) Normal tests, made in either air or water maintained, as nearly as practicable, at 21° cent. (70° Fahr.); and (2) Accelerated tests, made in air, steam or water, at temperature of 45° cent. (113° Fahr.) and upward. The Committee recommends that these tests be made in the following manner:

60.—*Methods.*—Pats, about 3 in. in diameter, ½ in. thick at the center, and tapering to a thin edge, should be made on clean glass plates (about 4 in. square) from cement paste of normal consistency, and stored in a moist closet for 24 hours.

61.—*Normal Tests.*—After 24 hours in the moist closet, a pat is immersed in water for 28 days and observed at intervals. A similar pat, after 24 hours in the moist closet, is exposed to the air for 28 days or more and observed at intervals.



BALL-BEARING BLOCK FOR TESTING MACHINE

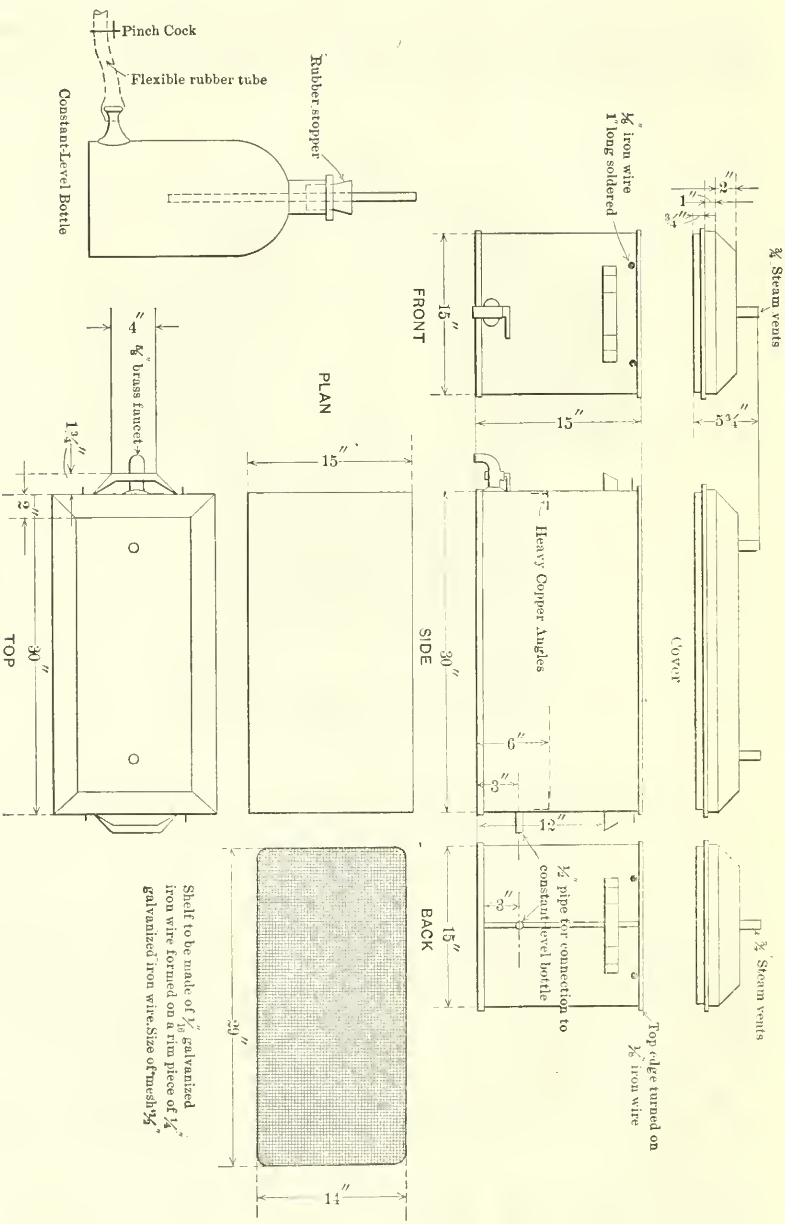
FIG. 7.

62.—*Accelerated Test.*—After 24 hours in the moist closet, a pat is placed in an atmosphere of steam, upon a wire screen 1 in. above boiling water, for 5 hours. The apparatus should be so constructed that the steam will escape freely and atmospheric pressure be maintained. Since the type of apparatus used has a great influence on the results, the arrangement shown in Fig. 8 is recommended.

63.—Pats which remain firm and hard and show no signs of cracking, distortion, or disintegration are said to be "of constant volume" or "sound."

64.—Should the pat leave the plate, distortion may be detected best with a straight-edge applied to the surface which was in contact with the plate.

APPARATUS FOR MAKING ACCELERATED TEST FOR SOUNDNESS OF CEMENT.



To be made of sheet copper weighing 22 oz. per sq. ft., tinned inside. All seams to be lapped where possible. Hard solder only to be used

FIG. 8.

65.—In the present state of our knowledge it cannot be said that a cement which fails to pass the accelerated test will prove defective in the work; nor can a cement be considered entirely safe simply because it has passed these tests.

GEORGE S. WEBSTER, *Chairman.*
RICHARD L. HUMPHREY, *Secretary.*
W. B. W. HOWE,
F. H. LEWIS,
S. B. NEWBERRY,
ALFRED NOBLE,
CLIFFORD RICHARDSON,
L. C. SABIN,
GEORGE F. SWAIN.

METHODS FOR TESTING CEMENT.*
CONDENSED FOR USE IN SPECIFICATIONS.

1. *Sampling.*

Cement in barrels shall be sampled through a hole made in the head, or in one of the staves midway between the heads, by means of an auger or a sampling iron similar to that used by sugar inspectors; if in bags, the sample shall be taken from surface to center. Cement in bins shall be sampled in such a manner as to represent fairly the contents of the bin. The number of samples taken shall be as directed by the Engineer, who will determine whether the samples shall be tested separately or mixed.

The samples shall be passed through a sieve having twenty meshes per linear inch, in order to break up lumps and remove foreign material.

2. *Chemical Analysis.*

The methods to be followed, except for determining the loss on ignition should be those proposed by the Committee on Uniformity in the Analysis of Materials for the Portland Cement Industry, reported in the *Journal* of the Society for Chemical Industry, Vol. 21, p. 12, 1902, and published in *Engineering News*, Vol. 50, p. 60, 1903, and in *Engineering Record*, Vol. 48, p. 49, 1903, and in addition thereto the following:

(*a*) The insoluble residue may be determined as follows: To a 1-gramme sample of the cement are added 30 cu. cm. of water and 10 cu. cm. of concentrated hydrochloric acid, and then warmed until effervescence ceases, and digested on a steam bath until dissolved. The residue is filtered, washed with hot water, and the filter paper and contents digested on the steam bath in a 5% solution of sodium carbonate. This residue is filtered, washed with hot water, then with hot hydrochloric acid, and finally with hot water, and then ignited at a red heat and weighed. The quantity so obtained is the insoluble residue.

(*b*) The loss on ignition shall be determined in the following manner: ½ gramme of cement is heated in a weighed platinum crucible, with cover, for 5 minutes with a Bunsen burner (starting with a low flame and gradually increasing to its full height) and then heated for 15 minutes with a blast lamp; the difference between the weight after cooling and the original weight is the loss on ignition. The temperature should not exceed 900° cent., or a low red heat; the ignition should preferably be made in a muffle.

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*Accompanying Final Report of Special Committee on Uniform Tests of Cement, dated January 17th, 1912.

3. *Specific Gravity.*

The determination of specific gravity shall be made with a standardized Le Chatelier apparatus. This consists of a flask (D), Fig. 1, page 677, of about 120 cu. cm. capacity, the neck of which is about 20 cm. long; in the middle of this neck is a bulb (C), above and below which are two marks (F) and (E); the volume between these two marks is 20 cu. cm. The neck has a diameter of about 9 mm., and is graduated into tenths of cubic centimeters above the mark (F).

Benzine (62° Beaumé naphtha) or kerosene free from water shall be used in making the determination. The flask is filled with either of these liquids to the lower mark (E) and 64 grammes of cement, cooled to the temperature of the liquid, is slowly introduced through the funnel (B), (the stem of which should be long enough to extend into the flask to the top of the bulb (C)), taking care that the cement does not adhere to the sides of the flask, and that the funnel does not touch the liquid. After all the cement is introduced, the level of the liquid will rise to some division of the graduated neck; this reading, plus 20 cu. cm., is the volume displaced by 64 grammes of the cement. The specific gravity is obtained from the formula,

$$\text{Specific gravity} = \frac{\text{Weight of cement, in grammes,}}{\text{Displaced volume, in cubic centimeters.}}$$

The flask, during the operation, is kept immersed in water in a jar (A), in order to avoid variations in the temperature of the liquid in the flask, which should not exceed $\frac{1}{2}$° cent. The results of repeated tests should agree within 0.01.

The determination of specific gravity shall be made on the cement as received; if it should fall below 3.10, a second determination shall be made after igniting the sample at a low red heat. The ignition shall be carried out in the following manner:

The flask, during the operation, is kept immersed in water in a jar (A) in order to avoid variations in the temperature of the liquid in the flask, which should not exceed $\frac{1}{2}$° cent. The results of repeated tests should agree within 0.01. The determination of specific gravity should be made on the cement as received; if it should fall below 3.10, a second determination should be made after igniting the sample in a covered dish, preferably of platinum, at a low red heat not exceeding 900° cent. The sample should be heated for 5 minutes with a Bunsen burner (starting with a low flame and gradually increasing to its full height) and then heated for 15 minutes with a blast lamp; the ignition should preferably be made in a muffle.

4. *Fineness.*

The fineness shall be determined by weighing the residue retained on No. 100 and No. 200 sieves. The sieves, 8 in. in diameter, shall be of brass wire cloth conforming to the following requirements:

No. of sieve.	Diameter of wire.	Meshes, per Linear Inch.	
		Warp.	Woof.
100	0.0042 to 0.0048 in.	95 to 101	93 to 103
200	0.0021 to 0.0023 "	192 to 203	190 to 205

The meshes in any smaller space, down to 0.25 in., shall be proportional in number.

Fifty grammes of cement, dried at a temperature of 100° cent. (212° Fahr.), shall be placed on the No. 200 sieve, which, with pan and cover attached, is held in one hand in a slightly inclined position, and moved forward and backward about 200 times per minute, at the same time striking the side gently, on the up stroke, against the palm of the other hand. The operation is continued until not more than 0.05 gramme will pass through in one minute. The residue is weighed, then placed on the No. 100 sieve, and the operation repeated. The work may be expedited by placing in the sieve a few large steel shot, which should be removed before the final one minute of sieving. The sieves should be thoroughly dry and clean.

5. *Normal Consistency.*

The amount of water, expressed in percentage by weight of the dry cement, required to produce a paste* of the plasticity desired, termed "normal consistency," shall be determined with the Vicat apparatus:

This consists of a frame (*A*), Fig. 2, page 680, bearing a movable rod (*B*), weighing 300 grammes, one end (*C*) being 1 cm. in diameter for a distance of 6 cm., the other having a removable needle (*D*), 1 mm. in diameter, 6 cm. long. The rod is reversible, and can be held in any desired position by a screw (*E*), and has midway between the ends a mark (*F*) which moves under a scale (graduated to millimeters) attached to the frame (*A*). The paste is held in a conical, hard-rubber ring (*G*), 7 cm. in diameter at the base, 4 cm. high, resting on a glass plate (*H*) about 10 cm. square.

In making the determination of normal consistency, the same quantity of cement as will be used subsequently for each batch in

* The term "paste" is used in these specifications to designate a mixture of cement and water, and the word "mortar" to designate a mixture of cement, sand, and water.

making the test pieces, but not less than 500 grammes, together with a measured amount of water, is kneaded into a paste, as described in Section 9, and quickly formed into a ball with the hands, completing the operation by tossing it six times from one hand to the other, maintained about 6 in. apart; the ball resting in the palm of one hand is pressed into the larger end of the rubber ring held in the other hand, completely filling the ring with paste; the excess at the larger end is then removed by a single movement of the palm of the hand; the ring is then placed on its larger end on a glass plate and the excess paste at the smaller end is sliced off at the top of the ring by a single oblique stroke of a trowel held at a slight angle with the top of the ring. During these operations care must be taken not to compress the paste. The paste confined in the ring, resting on the plate, is placed under the rod, the larger end of which is carefully brought in contact with the surface of the paste; the scale is then read, and the rod quickly released.

The paste is of normal consistency when the cylinder settles to a point 10 mm. below the original surface in one-half minute after being released. The apparatus must be free from all vibrations during the test.

Trial pastes are made with varying percentages of water until the normal consistency is attained.

Having determined the percentage of water required to produce a paste of normal consistency, the percentage required for a mortar containing, by weight, one part of cement to three parts of standard Ottawa sand, shall be obtained from the following table, the amount being a percentage of the combined weight of the cement and sand.

PERCENTAGE OF WATER FOR STANDARD MORTARS.

Neat.	One cement, three standard Ottawa sand.	Neat.	One cement, three standard Ottawa sand.	Neat.	One cement, three standard Ottawa sand.
15	8.0	23	9.3	31	10.7
16	8.2	24	9.5	32	10.8
17	8.3	25	9.7	33	11.0
18	8.5	26	9.8	34	11.2
19	8.7	27	10.0	35	11.3
20	8.8	28	10.2	36	11.5
21	9.0	29	10.3	37	11.7
22	9.2	30	10.5	38	11.8

6. *Time of Setting.*

The time of setting shall be determined with the Vicat apparatus in the following manner:

A paste of normal consistency is moulded in the hard-rubber ring, as described in Section 5, and placed under the rod (B), the smaller end of which is then carefully brought in contact with the surface of the paste, and the rod quickly released.

The cement is considered to have acquired its initial set when the needle ceases to pass a point 5 mm. above the glass plate; and the final set, when the needle does not sink visibly into the paste.

The test pieces must be kept in moist air during the test.

7. *Standard Sand.*

The sand shall be natural sand from Ottawa, Ill., screened to pass a No. 20 sieve, and retained on a No. 30 sieve.

The sieves shall be at least 8 in. in diameter, and the wire cloth shall be of brass wire and shall conform to the following requirements:

No. of sieve.	Diameter of wire.	Meshes, per Linear Inch.	
		Warp.	Woof.
20	0.016 to 0.017 in.	19.5 to 20.5	19 to 21
30	0.011 to 0.012 "	29.5 to 30.5	28.5 to 31.5

Sand which has passed the No. 20 sieve is standard when not more than 5 grammes passes the No. 30 sieve in one minute of continuous sifting of a 500-gramme sample.*

8. *Form of Test Pieces.*

For tensile tests, the form of test pieces shown in Fig. 3, page 683, shall be used.

For compressive tests, 2-in. cubes shall be used.

9. *Mixing and Moulding.*

The material shall be weighed, placed on a non-absorbent surface, thoroughly mixed dry if sand be used, and a crater formed in the center, into which the proper percentage of clean water shall be poured; the material on the outer edge shall be turned into the center by the aid of a trowel. As soon as the water has been absorbed, the operation of mixing shall be completed by vigorously kneading with the hands for one minute.

Immediately after mixing, the paste or mortar shall be placed in the mould (Figs. 4 and 5, page 683) with the hands, pressed in firmly with the fingers, and smoothed off with a trowel without ramming. The material shall be heaped above the mould, and, in smoothing off, the trowel shall be drawn over the mould in such a manner as to exert a moderate pressure on the material; the mould shall then be turned over and the operation of heaping and smoothing off repeated.

*This sand may now (1912) be obtained from the Ottawa Silica Co., at a cost of two cents per pound, f. o. b. cars, Ottawa, Ill.

The temperature of the room and of the mixing water shall be maintained as nearly as practicable at 21° cent. (70° Fahr.).

10. *Storage of the Test Pieces.*

During the first 24 hours after moulding, the test pieces shall be stored in a moist closet. This consists of a box of soapstone or slate, or of wood lined with metal, the interior surface being covered with felt or broad wicking kept wet, the bottom of the box being kept covered with water. The interior of the box is provided with glass shelves on which to place the test pieces, the shelves being so arranged that they may be withdrawn readily.

Test pieces from any sample which vary more than 3% in weight from the average, after removal from the moist closet, shall not be considered in determining strength.

After 24 hours in the moist closet, the pieces to be tested after longer periods shall be immersed in water in storage tanks or pans made of non-corrodible material.

The air and water in the moist closet and the water in the storage tanks shall be maintained, as nearly as practicable, at 21° cent. (70° Fahr.).

11. *Tests of Tensile Strength.*

The tests may be made with any standard machine.

The clip is shown in Fig. 6, page 685. It must be made accurately, the pins and rollers turned, and the rollers bored slightly larger than the pins so as to turn easily. There should be a slight clearance at each end of the roller, and the pins should be kept properly lubricated and free from grit. The clips shall be used without cushioning at the points of contact.

The test pieces shall be broken as soon as they are removed from the water. The load shall be applied at the rate of 600 lb. per minute.

Test pieces which do not break within $\frac{1}{4}$ in. of the center, or are otherwise manifestly faulty, shall be excluded in determining average results.

12. *Tests of Compressive Strength.*

The tests may be made with any machine provided with means for so applying the load that the line of pressure is along the axis of the test piece. A ball-bearing block for this purpose is shown in Fig. 7, page 687.

The test pieces as soon as they are removed from the water shall be placed in the testing machine, with a piece of heavy blotting paper on each of the crushing faces, which should be those that were in contact with the mould.

13. *Constancy of Volume.*

The tests for constancy of volume comprise "normal tests," which are made in air or water, maintained as nearly as practicable, at 21° cent. (70° Fahr.), and the "accelerated test," which is made in steam. These tests shall be made in the following manner:

Pats about 3 in. in diameter, ½ in. thick at the center, and tapering to a thin edge, shall be made on clean glass plates (about 4 in. square) from cement paste of normal consistency, and stored in a moist closet for 24 hours.

Normal Tests.—After 24 hours in the moist closet, a pat is immersed in water and observed at intervals. A similar pat, after 24 hours in the moist closet, is exposed to the air for 28 days or more and observed at intervals. The air and water are maintained, as nearly as practicable, at 21° cent. (70° Fahr.).

Accelerated Test.—After 24 hours in the moist closet, a pat is placed in an atmosphere of steam, upon a wire screen 1 in. above boiling water, for 5 hours, the apparatus being such that the steam will escape freely and atmospheric pressure be maintained. The apparatus is shown in Fig. 8, page 688.

The cement passes these tests when the pats remain firm and hard, with no signs of cracking, distortion, or disintegration.

George S. Webster, *Chairman.*
Richard L. Humphrey, *Secretary.*
W. B. W. Howe,
F. H. Lewis,
S. B. Newberry,
Alfred Noble,
Clifford Richardson,
L. C. Sabin,
George F. Swain.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1229

A REINFORCED CONCRETE INFILTRATION WELL AND PUMPING PLANT.*

BY FREDERICK N. HATCH, JUN. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. WILLIAM R. COPELAND, H. F. DUNHAM, AND FREDERICK N. HATCH.

In this paper will be given a brief description of the design, equipment, and construction of a pumping station recently constructed for the Chesapeake and Ohio Railway at Silver Grove, Ky., as a part of the terminal improvements carried out by the company with which the writer is connected.

It was estimated that the ultimate quantity of water which would be required to supply the terminal would not exceed 1 000 000 gal. per 24 hours, and that the demand would be at a fairly uniform rate throughout that period. As the terminal is near the bank of the Ohio River, water was to be obtained from that stream and delivered to two 100 000-gal. tanks on towers 45 ft. high.

The Ohio River at this point is subject to an extreme variation of about 69 ft. between low and high stages, and it was necessary to design a plant which would operate satisfactorily at any stage. It was also desirable that the plant should be as nearly automatic in its operation as possible, as it would have to be about 700 ft. from the nearest shop building.

* Presented at the meeting of May 1st, 1912.

Test borings, made at several points on the river bank, showed that the top soil is underlaid by sand and gravel, and that the river bed is in the same formation. With this in mind, it was decided to sink a well on the bank and provide openings in it so that it would receive water by infiltration from the river through the intervening sand and gravel. It was expected that water obtained in this way would not contain much suspended earthy matter, though the Ohio is a turbid stream during high-water stages.

The sounding taken at the point at which the well was finally located showed the different strata to be as follows, the elevation given being that of the top of each stratum:

	Elevation.
Extreme high water in the river	407.3
Surface of ground, loam and sand	376.0
Loam and clay	361.0
Gravel, with some sand	354.0
Sand, with some gravel	343.0
Extreme low water in the river	338.1
Fine white sand	330.0
Bed-rock	291.0

The bottom floor was fixed at Elevation 328.0, or 10 ft. lower than extreme low water, and the motor floor is above the highest stage of water.

The pumping requirements and conditions are as follows:

Minimum capacity	700 gal. per min.
Maximum static head	127 ft.
Minimum " "	66 "
Discharge	through 1 000 ft. of 8-in. cast-iron pipe.

The pumps are to be driven by electric motors taking 3-phase, 60-cycle, 440-volt current; the motors are to be controlled automatically by the water level in the tanks; and all equipment is to be in duplicate.

Constant-speed, centrifugal pumps were selected on account of the great variation of effective head, and the fact that pumps of this type have a low starting torque, which is favorable to automatically-controlled, alternating-current, motor drive. The pumps are 5-in., constant-speed (1 140 rev. per min.), top-suction, vertical, single-stage, centrifugal turbines, manufactured by Henry R. Worthington. At low-water stages each has a capacity of 700 gal. per min. against a total head of 142 ft., and requires 43.5 motor h.p. to operate it; at high water, against a total head of 104 ft., the discharge is 1 150 gal. per min., and the required horse-power is 50.5.

Each pump is driven by a Westinghouse 50-h.p. squirrel-cage type, induction motor, mounted on a cast-iron base on the motor-room floor. Each motor is controlled by a separate automatic starter, the two starters being mounted on one board and connected to the power line so that only one motor can be operated at a time. The solenoid switches of the starters are actuated by single-phase current controlled by a float-switch on one of the tanks. In order to prevent too frequent operation of the pumps, the float-switch is arranged so that it does not close until the water level has fallen to a point 5 ft. below the top of the tanks.

The shafts connecting the pumps to the motors are held in alignment by guide-bearings, adjustable in all directions, attached to rigid, built-up beams. The entire weight of the shaft and any possible unbalanced thrust of the pump impeller is carried by a marine-type thrust-bearing, mounted just below each motor. Flexible couplings prevent any of the weight of the main shaft from being transmitted to the motor rotor shaft and its separate thrust-bearing.

The entire shell of the well, including the motor-house and roof, is of reinforced concrete. The shell was designed to be sunk as an open caisson below the ground level. Sufficient reinforcement was provided to prevent the walls from pulling apart in case the upper part of the shaft should be held by the forms or the friction of the earth, while the lower part was free. Besides providing for the erection stresses, the walls and bottom were reinforced to withstand any possible unbalanced earth or water pressure.

The motor-room floor was designed for a live load of 400 lb. per sq. ft. plus the concentrated loads of the motors, shafts, etc.

Water is admitted to the well through 63 openings formed by pieces of 5-in., wrought-iron pipe extending through the walls of the well below the low-water line. The aggregate area of the openings is 8.75 sq. ft. These openings are shown on Fig. 1, the photograph having been taken just after the sinking of the shaft began. Inside the well these holes are enclosed by a steel-plate chamber designed to withstand the unbalanced hydraulic pressure on the inlet side during high-water periods, with the inside of the shaft dry. A 24-in. sluice-gate, to control the inflow of water, is mounted on the side of this chamber. The gate is operated by a geared stand on the motor-room floor. To insure the proper spacing of the inlet chamber anchor-bolts, and also to provide an even joint surface, a steel angle companion flange was made with the flange of the chamber as a template. This companion flange was built into the concrete wall, and the anchor-bolts, which had enlarged ends tapped inside for 1-in. tap-bolts, were held in place by tap-bolts extending through the forms. A sheet-lead gasket was inserted between the companion flange and the flange of the chamber.

There is an opening, enclosed by a pipe railing, in the motor-room floor to give access to the lower part of the well, and steel ladders extend from bottom to top, inside.

The built-up beams for supporting the shaft guide bearings were designed for rigidity, and were set in pockets left in the walls of the well during construction. As the cover-plate serves as a walkway for inspecting the guide bearings, each beam has a pipe railing along one side of it.

A single I-beam, suspended above the motor-room floor and the hatchway, and extending as a cantilever beyond the entrance platform, serves as a track for a 4 000-lb. trolley provided to handle any of the heavy equipment.

An outside spiral steel stairway gives access to the head-house. The brackets supporting the stairway were built into pockets in the outside of the walls of the shaft.

The estimated weight of the well complete with its equipment is 530 tons, while the gross buoyancy during extreme high water, with no water inside the well, is about 465 tons, leaving a margin of stability of 65 tons, if the friction between the earth and the lower part of the shell is neglected.

Fig. 2 is a view of the completed well as seen from the land side. Fig. 3 shows the general design of the well and the equipment; but minor details have been omitted, and, to avoid confusion, some parts have been shown out of the true section.

Construction.—After leveling off the site, the steel-plate, cutting shoe was set up, and the forms were erected over it. The forms were of $\frac{7}{8}$-in. sheathing nailed to waling pieces cut to radius. Those for the

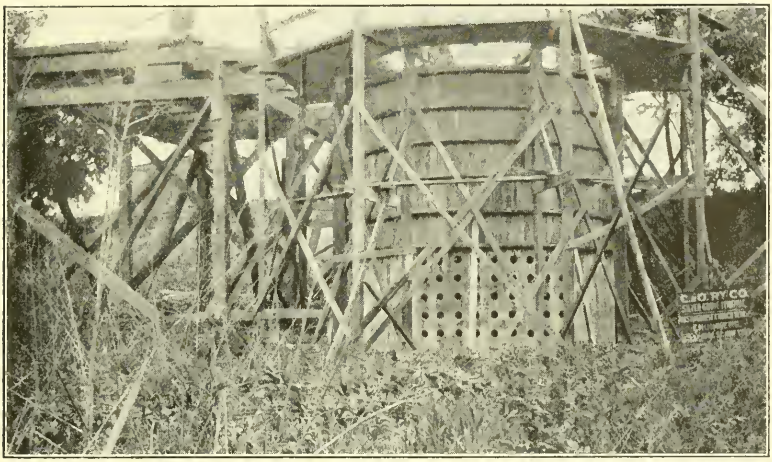


FIG. 1.—LOWER PART OF SHAFT BEFORE SINKING, SHOWING OPENINGS THROUGH WHICH WATER IS ADMITTED.

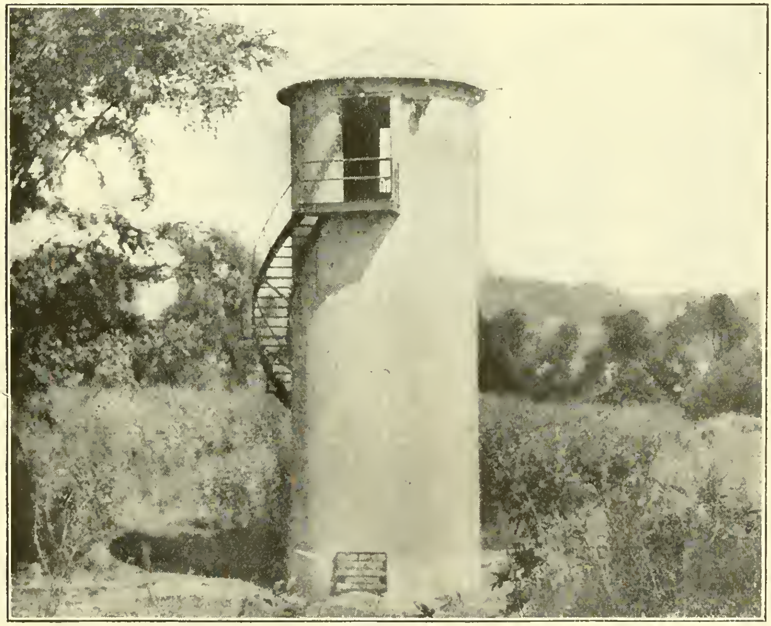


FIG. 2.—COMPLETED INFILTRATION WELL.

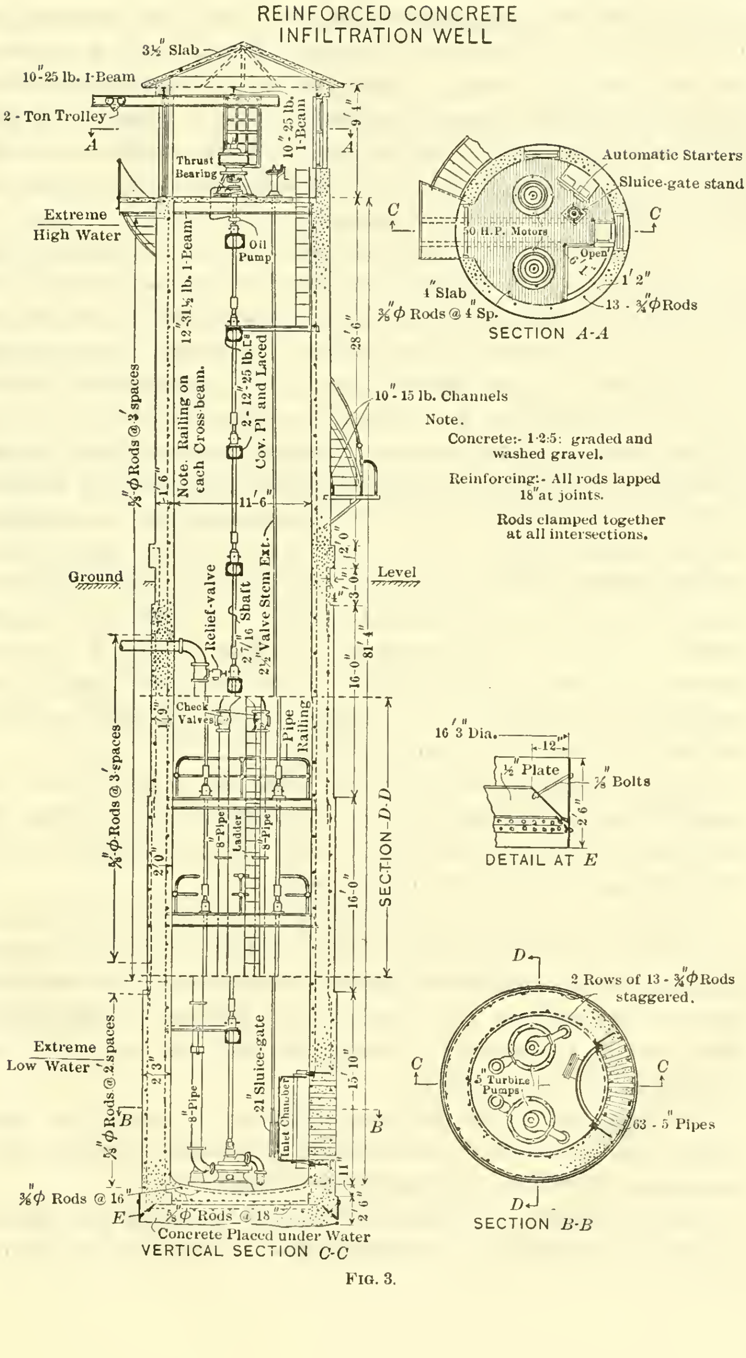

REINFORCED CONCRETE INFILTRATION WELL
3½" Slab
10"-25 lb. I-Beam
2 - Ton Trolley
Thrust Bearing
10"-25 lb. I-Beam
Extreme High Water
Oil Pump
12"-31½ lb. I-Beam
2 - 12"-25 lb. E's Cov. Pl and Laced
Note. Railing on each Cross-beam.
⅜" φ Rods @ 3' spaces
11'-6"
28'-6"
10"-15 lb. Channels
Ground
Level
Relief-valve
2 7/16" Shaft
2½" Valve Stem Ext.
81'-4"
16'-0"
Check Valves
Pipe Railing
8"-Pipe
Ladder
8"-Pipe
SECTION-D-D
Extreme Low Water
⅜" φ Rods @ 2' spaces
8"-Pipe
21" Sluice-gate
Inlet Chamber
15'-10"
⅜" φ Rods @ 16"
⅜" φ Rods @ 18"
Concrete Placed under Water
VERTICAL SECTION C-C
Automatic Starters
Sluice-gate stand
50 H.P. Motors
Open
4" Slab
⅜" φ Rods @ 4" Sp.
13 - ⅜" φ Rods
SECTION A-A
Note.
Concrete:- 1·2·5: graded and washed gravel.
Reinforcing:- All rods lapped 18" at joints.
Rods clamped together at all intersections.
16' 3" Dia.
½" Plate
⅞" Bolts
DETAIL AT E
2 Rows of 13 - ⅜" φ Rods staggered.
5" Turbine Pumps
63 - 5" Pipes
SECTION B-B

FIG. 3.

outside of the wall were supported by the posts of the working platform, and were held together, in sections, by steel bands arranged so that they could be loosened to allow the wall to slip through them. The inside forms were hung from cantilever brackets on the working platform, and were held in position by removable cross-braces, a construction which permitted the free use of the entire interior during the periods when excavating was being done.

The reinforcing rods were put in position, and a section of concrete about 5 ft. high was carefully placed and allowed to set. The forms were then loosened, and the material was excavated from the inside by a $\frac{3}{4}$-yd. orange-peel bucket handled by a derrick. As the excavation progressed the wall gradually sank. The operations of placing concrete and excavating were carried on alternately until the foot of the shaft reached its final position. The shell sank very freely through the earth, and it became necessary to provide some means of checking its descent at the proper place. This was accomplished by forming a concrete collar, integral with the wall, just above the ground level, and then taking a part of the load off the cutting shoe by blocking between this collar and the ground surface. The collar was designed to carry a large part of the suspended weight of the well if necessary. By the use of the collar the sinking of the wall was stopped at the proper place, and there was no settlement while the sand was being removed from beneath the cutting shoe.

Before the shaft reached the water level, the intake chamber was bolted in position and the sluice-gate was attached to it. When the excavation reached the water level, a large pulsometer pump was used to remove part of the water and fine sand.

As the flow of water through the open bottom of the shaft was too great to be handled by pumps, a diver was sent down to level off the bottom of the excavation and place the reinforcement for the rough bottom of the well; he also distributed the concrete for this bottom. After allowing the concrete to harden, the well was pumped dry and the finished bottom was put in.

The upper part of the shaft was constructed by raising the forms by stages as the wall was completed, and no conditions unusual to the construction of similar structures were encountered.

After the shaft was completed, the steel, guide-bearing supports were placed, leveled up, and grouted in place. Great care was required in getting the supports and bearings perfectly level and plumb, so that the vertical shafts would run smoothly.

This pumping plant has been in operation for some time, and it has been found, as anticipated, that the water reaching the interior of the well is free from suspended earthy matter, although at times the river has been quite turbid. At no time has there been a shortage of water in the well, even when the pump was running at its rated capacity and the river stage was very low.

The construction of that part of the well below the ground level was accomplished in 18 working days, and the entire part above ground in 6 working days.

As the construction of the well was only a small part of the work done at Silver Grove, and as the entire job has not been closed up, only approximate costs can be given. The figures show the actual costs of construction, and do not include engineering, drafting, and other overhead charges.

Grading and excavation................370 cu. yd. at $1.65
All concrete, including reinforcement,
material.........................280 " " " 3.15
All concrete, including reinforcement,
labor...........................280 " " " 3.85
All forms, material, and labor..........at $2.85 per cu. yd.
All steelwork, fabricated and otherwise,
material.........................28 000 lb. at $0.052
All steelwork, fabricated and otherwise,
labor...........................28 000 " " 0.022
(Costs of steel include painting.)
All equipment, piping, wiring, etc., in place..........$6 200

This pumping station was designed, constructed, and equipped by Westinghouse, Church, Kerr and Company, of New York.

DISCUSSION

Mr. Copeland.

WILLIAM R. COPELAND, ASSOC. AM. SOC. C. E. (by letter).—Mr. Hatch states that the well described in his paper "would receive water by infiltration from the river through the intervening sand and gravel." He adds: "the water reaching the interior of the well is free from suspended earthy matter, although at times the river has been quite turbid." He evidently infers that the water in the well filtered in from the river, but the writer believes that only a small part comes from the flowing water above the bed.

Most of the water in the well is probably ground-water, and the reasons for this are, on the one hand, that the "free flowing" water in the river is separated from the gravel and sandy sponge surrounding the well by a layer of almost impervious sediment on the river bottom; and, on the other hand, that there is a well-defined current of ground-water flowing from the highlands of the water-shed toward the river. Very few engineers realize how impervious the natural, silt-covered bottom of the Ohio is; but in a series of tests made on gravel bars lying in its bed, within 200 miles of Silver Grove, water drawn from a gravel stratum, less than 5 ft. below the silt, contained 75% of ground-water.

The presence of ground-water can be detected by analysis, as it has the following characteristics: First, it is clear; second, it is always cooler in summer and warmer in winter than surface river water; third, it is generally harder than the normal river waters of the East; fourth, it contains more iron in solution that the normal river waters of the East; and, finally, one often finds more free carbonic acid in ground-water than in river water.

Recent analyses have shown that the hardness found in the Ohio River water compared with that from the gravel bars referred to was 3 grains per gal. for the river as against 7 grains for the ground-water. The iron in the river water was less than 0.1 part per million, while the ground-water contained from 1 to 5 parts. The free carbonic acid in river water rarely exceeds 3 or 4 parts per million, but well and ground-waters often contain from 5 to 10, or even 15, parts per million.

These unfavorable chemical characteristics can be overcome in water softening by chemical treatment, but there is another feature of driven-well supplies which is more serious. As a general proposition, water moves slowly through the ground. Therefore, the suction caused by pumping creates currents which flow through the earth at abnormal rates. In this case, for instance, Mr. Hatch states that the pumps will draw water from the well at a rate of 700 gal. per min. In order to keep up this supply, water must flow into the well through pipes having a combined area of 8 sq. ft.—or at a rate of 80 gal. per sq. ft. per min. Engineers who have charge of water filters know that mechanical filters do not run at one-twentieth of this rate, nor sand filters one-hundredth part as fast. Mr. Copeland.

As a matter of fact, the water coming into the well must flow through the layer of sand contiguous to the openings in the casing at such speed that the currents in the little channels between the sand grains will dislodge particles of silt or sand from the sides of the channels and pack them into the voids near the well. As a result, the channels will choke up, and the yield of water will decrease.

The writer recalls a case where a contractor was to furnish to a city 3 000 000 gal. of water from wells sunk in the gravelly banks of a river. He drove the wells successfully, but got only 2 000 000 gal. the first year. The second year he put in a new series of wells about a mile away. These gave about 1 500 000 gal. per day, instead of 2 000 000, and every year thereafter the wells furnished smaller and smaller volumes until they were abandoned.

Engineers will do well to remember, therefore, that water obtained by such installations as that described by Mr. Hatch comes from the ground-water supply, rather than from the open channel of the river; that the hardness, carbonic acid, and iron content will increase, making it less favorable for boiler purposes, and that the yield may decrease year by year.

H. F. Dunham, M. Am. Soc. C. E. (by letter).—There are excellent features in this brief paper. The clear statement of facts relating to the construction, and the included costs, with efficiency data, are valuable contributions; but further information relating to the source and volume of supply would be of interest. Mr. Dunham.

When a well is sunk beside a river it is often difficult to prove that a large proportion of the supply comes from the river. If the sand and gravel beds penetrated by the well are extensive, it may be suspected, and in some cases it has been proved, that during a greater part of the time, and especially when the river is falling, the supply is water intercepted from land areas on its way to the river. Instances are recorded where a similar well, its connected machinery not being in operation, received quantities of water by overflow at the top. In such cases the river water finds a short cut—an artificial channel—through the well to partly exhausted gravel or sand beds which would be filled more slowly under natural conditions.

Wells supplied in large part from the land side are more satisfactory in one particular, for the tendency of silt to fill the interstices in the sand between the river and the well or near the well is lessened. Records should be kept showing the difference in elevation between the water level in the well when certain definite quantities are being supplied and the extreme low water in the river at that time. Year by year the record indicates the extent of the silting up of the beds.

Mr. Dunham.

Two or three changes may be suggested in the design to insure the following advantages:

1. Provision for increasing the supply and for restoring the original conditions if the present supply diminishes.

2. A reduction in the amount of electrical energy lost in friction.

3. Better opportunities for attention to machinery and for repairs.

By these items attention is drawn to estimates for a modified structure or housing in place of the cylindrical well, a regular intake supply main of proper size, and displacement pumps run by electric motors. The foundation for such a structure would be but little below low water.

The supply main, at about the same elevation, should be laid on a slightly descending grade, with branches to connect with vertical strainer wells which could be shut off for cleaning, repairs, or for an extension of the system when necessary.

The pumps should be set with their discharge valves below the level of the intake, and should be under automatic control. Then all interior parts would be accessible without the use of ladders, and there would be no trouble from water. The friction or loss of energy would be reduced from its present 70% to 25% or less, thus reducing the cost of electric current by from $300 to $400 per year, at a low estimate per kilowatt. The first cost would be differently apportioned to the various parts, but the total should not exceed the cost figures given by the author.

Mr. Hatch

Frederick N. Hatch, Jun. Am. Soc. C. E. (by letter).—Mr. Copeland and Mr. Dunham have brought out a few points which were not considered at length in the paper on account of a desire to confine the matter to a description of a rather novel method of constructing a well for obtaining a relatively small supply of water. The fact that these points were not taken up in detail must not be considered as an indication that they were not fully considered at the time the plant was designed.

The statement that the well "would receive water by infiltration from the river through the intervening sand and gravel" was not intended to convey the idea that the flow from the river to the well would take place by direct filtration along the shortest path at all times; during high-water stages of the river, it was expected that a large part of the water entering the well would come from the surface flow of the river by direct infiltration, and prolonged observation since the well was completed seems to indicate that such is the case. It was fully realized that such direct infiltration would not take place in a marked degree during low-water stages, but it was expected that a certain amount of the surface flow would reach the well along with the subterranean flow, because of filtration through the bed of the stream at some more or less distant points.

Mr. Hatch.

In regard to the impurities in solution in the water, it may be of interest to note the difference in the quality of the surface flow of the river and the true ground-water at this point, as indicated by analyses of two samples, taken at the same time and analyzed by the same chemists. The results are given in Table 1.

TABLE 1.—PROBABLE CONSTITUTION OF THE INCRUSTING SOLIDS.

In grains per United States gallon.

	Sample No. 5. from a test well after 24 hours' pumping.	Sample No. 6, from the river.
Silica	0.70	1.46
Alumina and oxide of iron	0.13	0.34
Carbonate of lime	11.70	1.40
Carbonate of magnesia	2.60	1.17
Sulphate of lime	2.30	5.86
Sodium chloride	0.50	1.90
Total solids	17.69	13.45
Hardness calculated as calcium carbonate	14.38	6.38
Reaction	Alkaline	Alkaline

As to the danger of the inflow being gradually reduced by the silting up of the interstices in the surrounding sand, it may be said that such a possibility was recognized, and it would in no way prove serious in this case, as it would then be only necessary to place a crib in the river and make a direct intake to the well with cast-iron pipe. The total cost of the plant, including such an intake, would not exceed the cost had the well been located originally where it would receive water directly from the river, and, until such a condition causes trouble—if it ever does—the water obtained is of better quality than would be received if it were taken directly from the river, and the fixed charges on the investment are less.

In considering other methods of operation of the pumping plant than the one selected and described in the paper, it must be borne in mind that the water level of the river may vary as much as 69 ft., and that the banks are submerged at high-water stages. Any pump housing would be required to provide for those conditions. If electrically-driven displacement pumps were set a little below low water—as the writer understands Mr. Dunham to suggest—the housing of them would be a more serious problem than was the housing of the centrifugal pumps, because of the necessity of keeping the driving motors dry. Mr. Dunham states that with his scheme there would be no trouble from water; but it should be noted that all his vertical strainer wells would be inaccessible for many months of the year on account of the water of the river standing over their tops.

Mr. Hatch.

It is not clear how Mr. Dunham proposes to make such a large saving of power over the present arrangement, or how he arrives at the figure, 70%, which he states to be the loss of energy at present. It is obvious, however, that, with the same electric power, an electric motor driving a centrifugal pump will be just as efficient as one driving a displacement pump; it is also apparent that the energy required to overcome the static pressure and friction head in the discharge pipe will be the same in either case. The only remaining source of lost energy is in the pump, and the writer does not know of any type of displacement pump which would show an energy loss of only 25% where the loss with a centrifugal pump would be as much as 70 per cent.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1230

A FOUR-TRACK, CENTER-BEARING, RAILROAD DRAW SPAN.*

By Louis H. Shoemaker, M. Am. Soc. C. E.

With Discussion by Messrs. A. H. Markwart, Howard J. Cole, L. J. Le Conte, and Louis H. Shoemaker.

The four-track draw span recently completed for the Bessemer and Lake Erie Railroad at Conneaut Harbor, Ohio, is of interest mainly on account of the special type of construction necessitated by the extremely limited height from base of rail to masonry. It is somewhat unusual to be required to support a draw span 235 ft. long and 67 ft. wide, with a turning load of nearly 1 400 tons, in a depth of 5 ft. 8 in. Moreover, the frequent high-water stage made it necessary to limit the depth of the floor system to 3 ft. 3 in. from base of rail to underclearance.

To conform to these conditions, in a bridge designed for Cooper's E-60 engine loading, necessitated short panels and floor-beams of single track length. A radical departure from standard types of construction was evidently necessary. The unusual width of the structure, together with the limited height available for the drum and the distributing girders, evidently excluded from consideration the rim-bearing type of center. It was evident, also, that the practicability

* Presented at the meeting of April 3d, 1912.

of using the center-bearing type depended on the adoption of a design which would reduce the length of the center supporting girders to a minimum.

The general features of the design adopted to meet these conditions are as follows:

There are two main trusses of unusual depth, 32 ft. 7 in. apart from center to center. Two tracks are supported between the trusses and two on overhangs outside. The trusses are of the sub-divided Warren type, with 14 ft. $8\frac{1}{4}$-in. panels. Deep overhead transverse trusses at the main panel points, with cantilever extensions, support the floor system by three lines of hangers. At the sub-panel points the floor system is supported from three lines of longitudinal trusses, one line overhead on the center line of bridge and two lines on the outside, which in turn are supported by the transverse trusses. There are two pairs of center supporting girders, the ends of each pair being connected by short girders, which support the main trusses, distributing the dead load from the main trusses to the transverse supporting girders, and also transmitting the live load from the main trusses through the center wedges to the masonry. The main carrying girders are supported by eight 12 by $1\frac{7}{8}$-in. eye-bars from a short longitudinal double-web girder which is supported on the center.

The bridge turns on a 34-in. phosphor-bronze disc acting between two nickel-steel discs. The pressure on these discs is about 3 500 lb. per sq. in. There are four end and two center wedges, and the power for operating them is transmitted by a longitudinal shaft, on the center line of the bridge, placed several feet above the track level and supported on the center suspenders. All wedges are driven transversely to the center line of the bridge.

The turning operation is performed by two pairs of pinions connected by equalizers acting on a rack having a pitch diameter of 41 ft. $2\frac{3}{16}$ in. The ends of the bridge are fitted with rail locks. Figs. 1 and 2 and Plate XII show the general construction and the arrangement of the machinery in detail. As it is not necessary to turn the bridge at present, the motive power has not yet been installed.

The views, Figs. 3 and 4, were taken during a loading test of the bridge. Fig. 3 shows one overhang loaded with heavy freight engines, the average weight being 6 100 lb. per lin. ft. of track. The maximum deflections observed were entirely satisfactory.

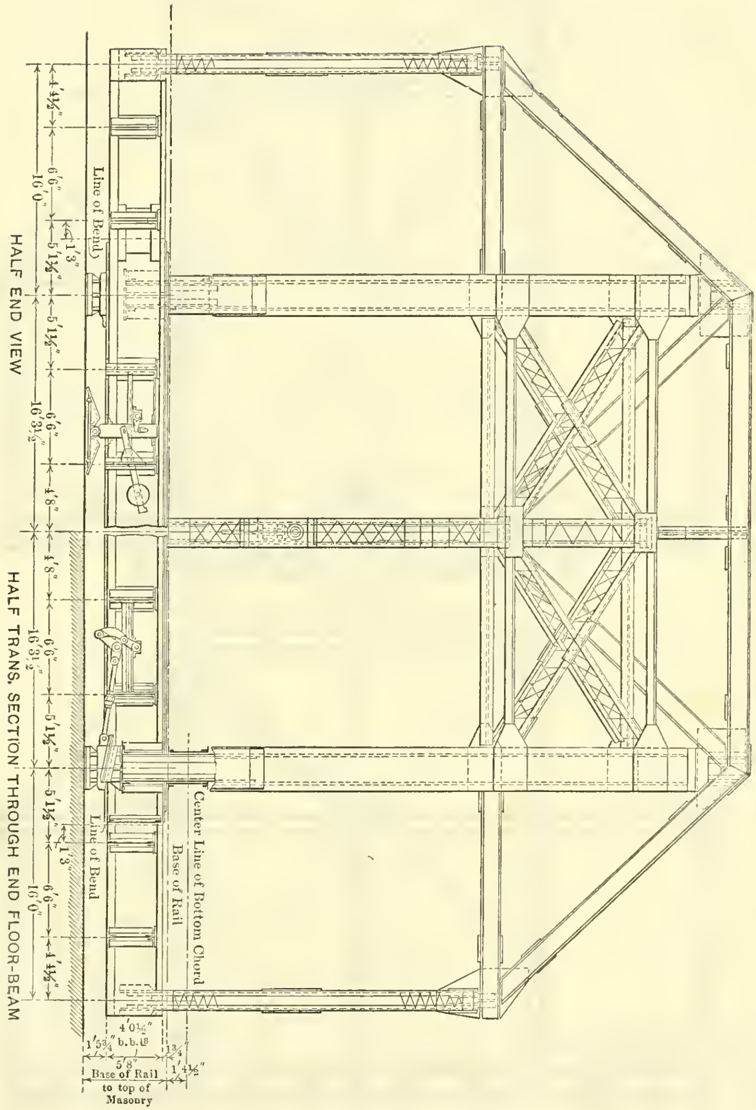
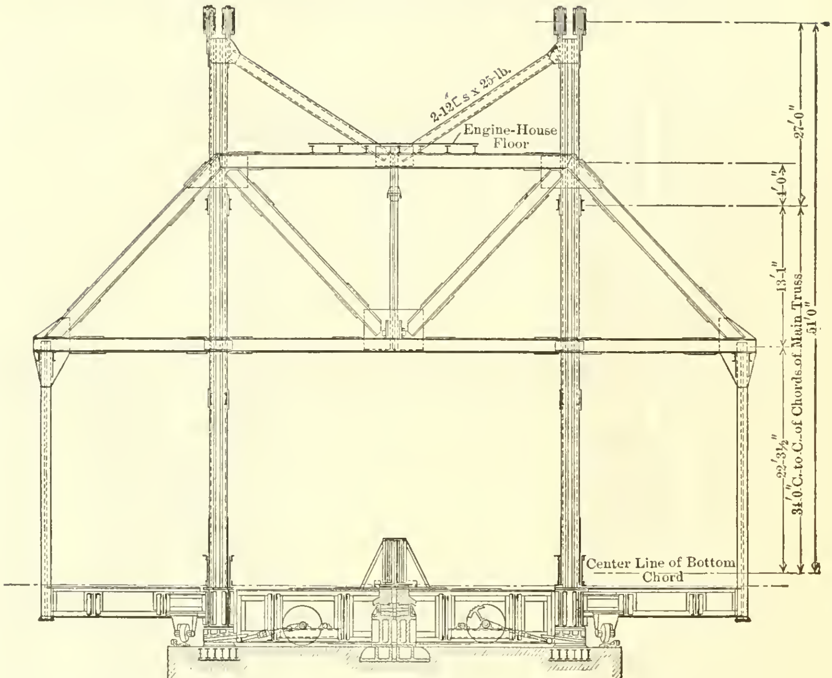
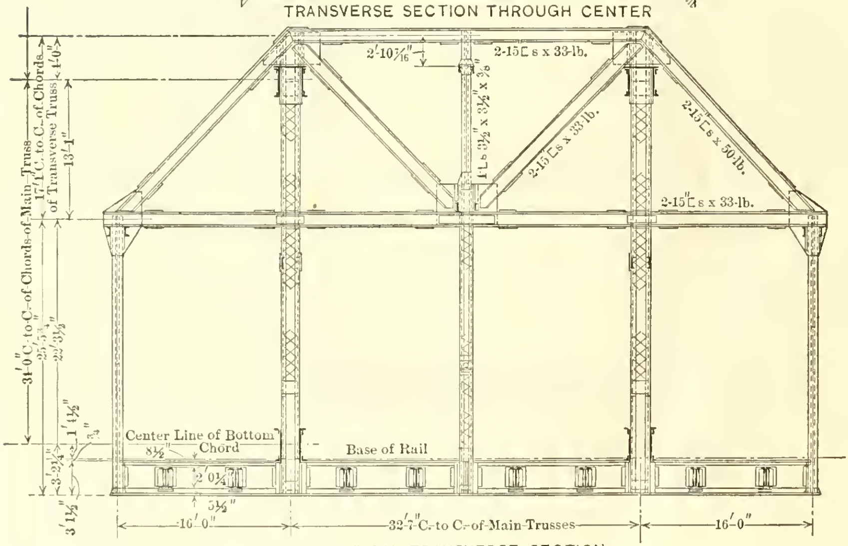

HALF END VIEW
HALF TRANS. SECTION THROUGH END FLOOR-BEAM
Line of Bend
Line of Bend
Center Line of Bottom Chord
Base of Rail
16′0″
16′3½″
16′3½″
16′0″
4′4½″
6′6″
5′1½″
5′1½″
6′6″
4′8″
4′8″
6′6″
5′1½″
5′1½″
6′6″
4′4½″
1′3″
1′3″
4′0½″ b.b. ∟s
1′5¾″
5′8″
Base of Rail to top of Masonry
1′4½″

FIG. 1.



TRANSVERSE SECTION THROUGH CENTER

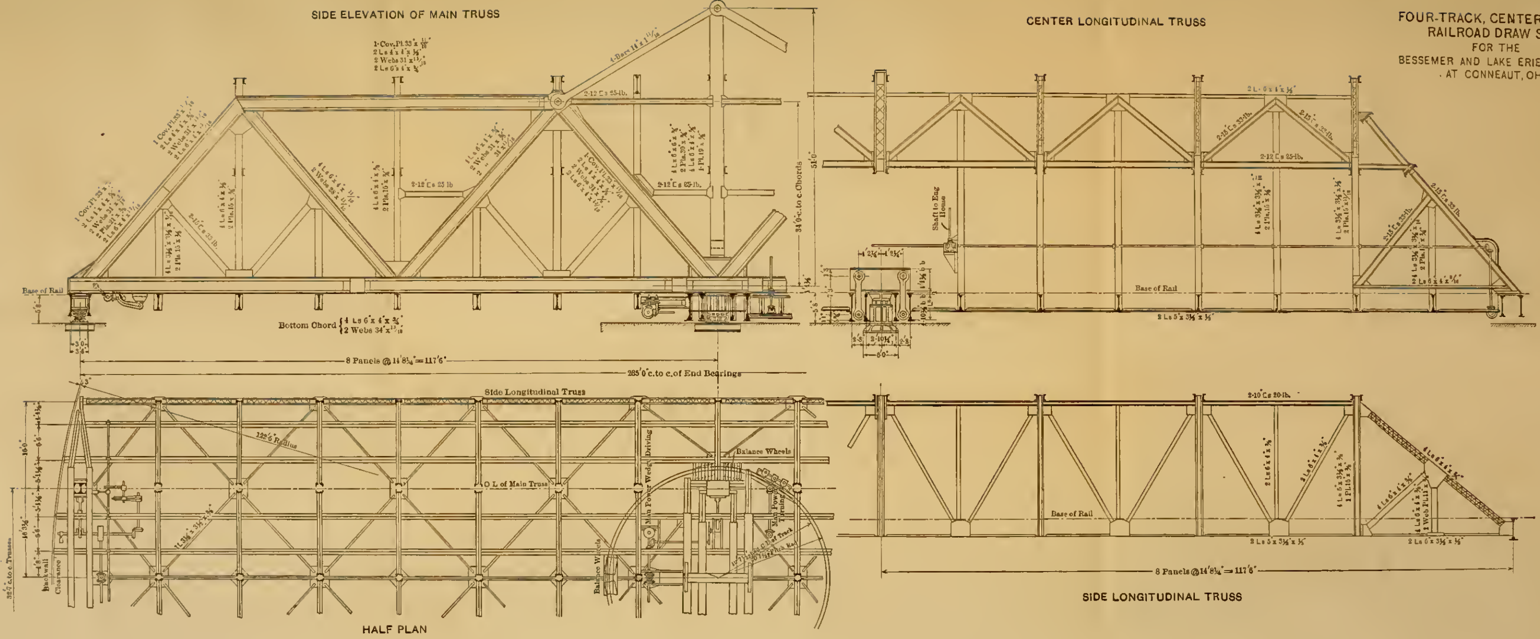


TYPICAL TRANSVERSE SECTION

FIG. 2.

PLATE XII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXV, No. 1230.
SHOEMAKER ON
RAILROAD DRAW SPAN.

FOUR-TRACK, CENTER-BEARING
RAILROAD DRAW SPAN
FOR THE
BESSEMER AND LAKE ERIE RAILROAD
AT CONNEAUT, OHIO.



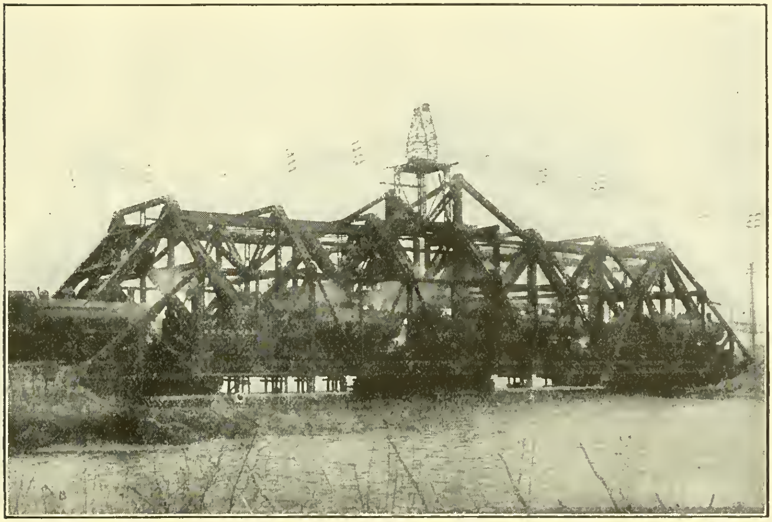


FIG. 3.—FOUR-TRACK, CENTER-BEARING, RAILROAD DRAW SPAN AT CONNEAUT HARBOR, OHIO, DURING LOADING TEST.

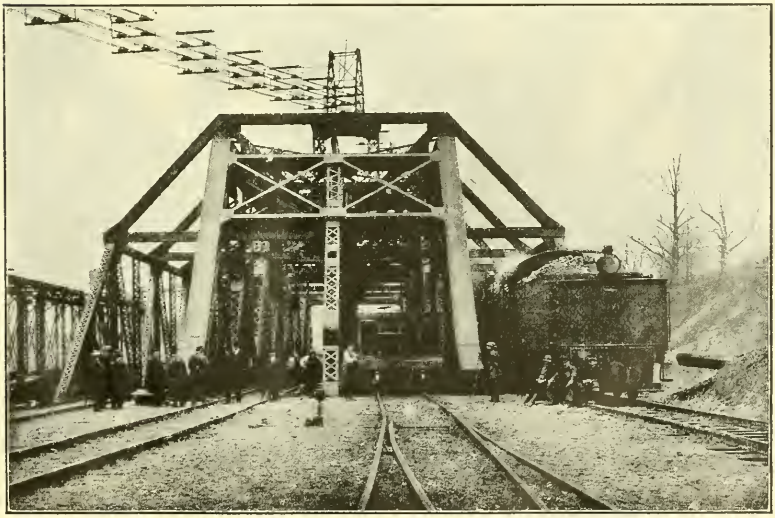


FIG. 4.—END VIEW OF CONNEAUT BRIDGE DURING LOADING TEST.

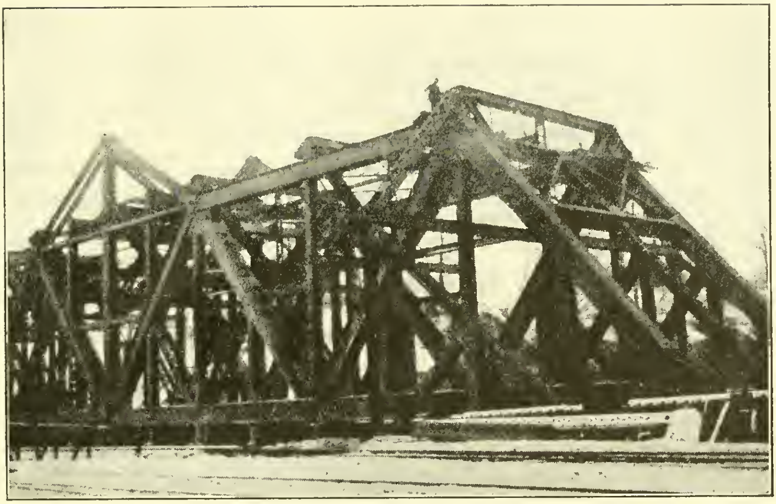


Fig. 5.—Conneaut Draw Span During Erection. Commencing to Place the Overhang.

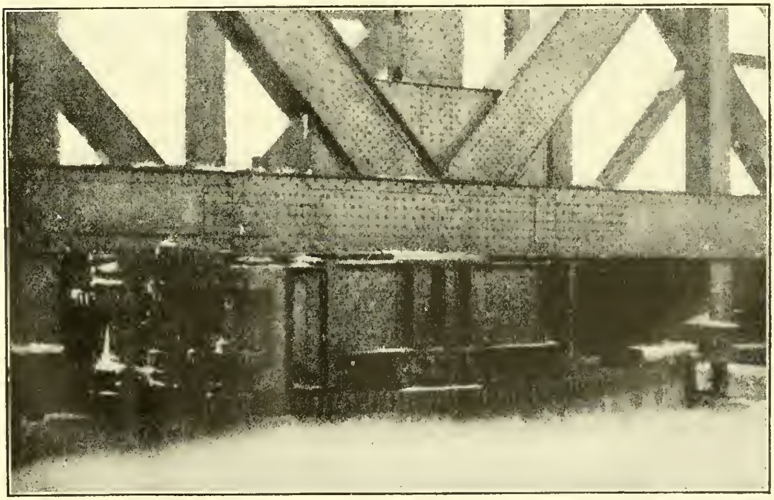


Fig. 6.—Conneaut Draw Span. Center Carrying Girders, and Rack and Pinions.

Fig. 5 shows the structure during erection, when the placing of the overhangs had just begun. Fig. 6 shows the central carrying girders and the rack and pinions.

The structure was calculated for Cooper's E-60 live load on four tracks, by the American Railway Engineering and Maintenance of Way Association Specifications. It was designed in the Designing Office of the American Bridge Company, at Pittsburgh, Pa., and fabricated and erected by them for the Bessemer and Lake Erie Railroad Company, H. T. Porter, M. Am. Soc. C. E., Chief Engineer.

DISCUSSION

Mr. Markwart. A. H. Markwart, Assoc. M. Am. Soc. C. E. (by letter).—Mr. Shoemaker's brief record of this draw-span is of considerable interest to bridge engineers. Structures of this type, which present unusual problems, are well worth describing, and often furnish an interesting precedent to engineers having problems of the same sort.

This structure is novel in four particulars, namely, the use of two main supporting trusses, heavy loading, limited height from base of rail to masonry, and limited distance from base of rail to bottom of chords.

In the case of four-track, railroad swing bridges, it is usually the custom to have three trusses with two tracks on each side of the center truss; also, in many notable designs, it has been found that great economy could be effected by having only two trusses with two tracks between them, the floor-beams having cantilever ends for the support of the other two tracks.

The designers of this bridge undoubtedly found it impossible to support the outside tracks by cantilevers from the ends of the floor-beams, on account of the shallow depth of the latter. This being the case, a third supporting truss between the two main trusses offered no advantage, except to make it possible to design the interior floor-beams to suit the available depth.

However, the design adopted has all the advantages of the cantilever scheme, as well as those of a third main supporting truss. The support of the ends of the outside beams and the reduction of the span of the inside beams have enabled the designers to provide all floor-beams inside the allowable depth.

While the bridge is not particularly sightly, from an esthetic standpoint, the designers are to be congratulated in that they have successfully planned the structure to conform with the unusual clearance conditions.

Mr. Cole. Howard J. Cole, M. Am. Soc. C. E.—The speaker desires to call attention to the fact that the first four-track draw-span in America was built over the Harlem River in New York City, about fifteen years ago, by the New York Central and Hudson River Railroad, Walter Katté, M. Am. Soc. C. E., Chief Engineer.

It has three trusses, 389 ft. from center to center of end pins and 26 ft. apart in the clear, carrying the four tracks on a ballasted floor.

There was no limiting depth, in this case, between the base of the rail and the top of the masonry, but the United States Government required 24 ft. clearance above high water. This draw-span is notable for its weight, 2 500 tons.

Mr. Le Conte. L. J. Le Conte, M. Am. Soc. C. E. (by letter).—The writer is much pleased with this new design for a center-bearing draw-bridge, particularly because, for many years past, he has had a strong prejudice against all center-bearing devices. Mr. Le Conte.

The general scheme of doing away with the old-fashioned center tower, and bringing both sets of main inclined posts to one point over the center of the center pier is commendable. The heavy compound box-girder, which takes the place of a floor-beam at this site, is designed to carry the entire dead load of the bridge—some 1 400 tons—when swinging open. This is all simple enough, but when it comes to supporting this heavy compound box-girder on a center-pintle bearing, the serious side of the problem is brought boldly to the front.

The writer regrets that the plans submitted do not show more of the useful details, as that would probably permit of fair criticism of the design; nevertheless, the details are sufficient to indicate a doubt as to the ability of the design to stand up permanently under the dead load strains when the draw-span is on the swing. The desired details are lacking particularly in the case of the small cross-girders and eye-bars, which are designed to carry the entire dead load, also, the packing of the joints and girder connections, all of which it is very important to know in order to make fair discussion possible. Of course, it is natural to presume that the design is all right in its details, but the plans ought to show enough to establish the fact with reasonable certainty. It is very important that all parts of the compound box-girder and the little overhead cross-girders should be designed so as to be easily inspected and painted at any time that it may be advisable or necessary.

The center-bearing device, of course, has one great advantage which cannot be over-estimated, namely, the ease with which it turns. If this design proves to be a success, it will be noted as an important step in the right direction.

Louis H. Shoemaker, M. Am. Soc. C. E. (by letter).—As Mr. Le Conte expresses a doubt as to the possibility of supporting the load of 1 400 tons in a permanent manner by the method described in the paper, the writer submits Fig. 7, a detail of the center casting and longitudinal supporting girder, which he trusts will explain the construction and remove all doubts as to its sufficiency. Mr. Shoemaker.

By distributing the load over four transverse girders, it was possible to keep the flange sections and rivet grips well within the limits of good practice. This arrangement also adapted itself to the best possible device for supporting from the center casting, that is, with eye-bars and pins, by which a practically perfect distribution of load through girders and hangers is secured.

Attention is called to the superiority—in the above respect as well as in that of accessibility—of this design over the usual method of supporting heavy draw-bridges from center bearings, in which two transverse supporting girders, with bolt girders framed between them

Mr. Shoemaker.

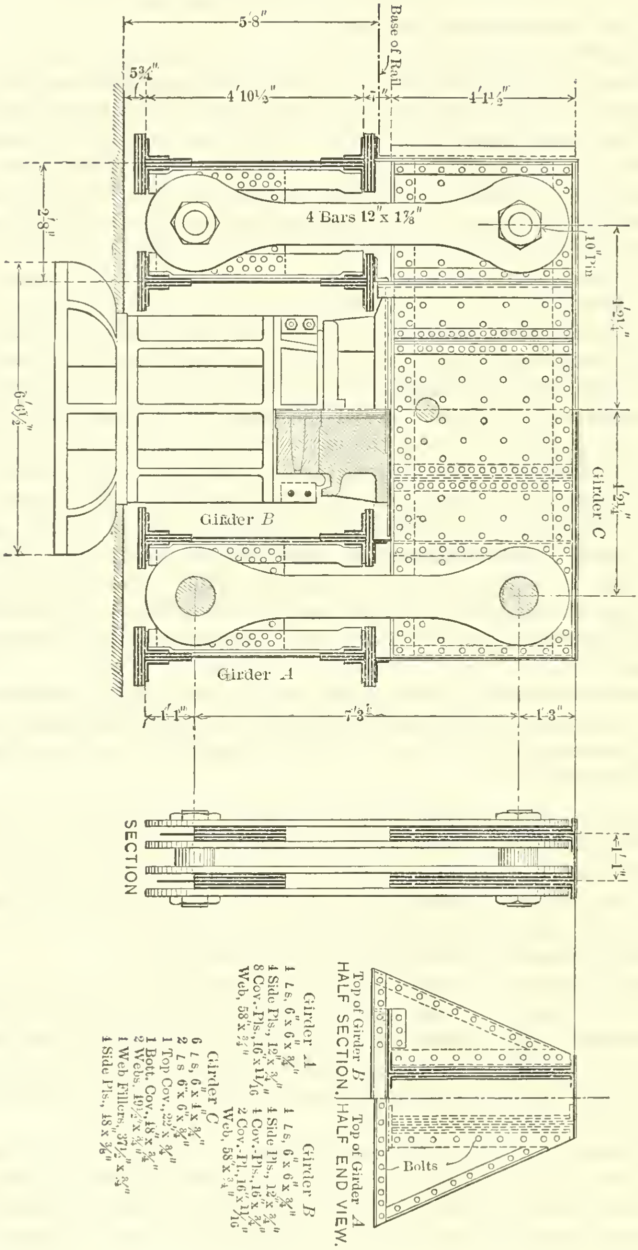


Fig. 7.

Mr. Shoemaker.

as close as possible to the center casting, are hung by a number of large round rods from a broad saddle girder. The design of the longitudinal supporting girder, *C*, was rendered comparatively easy by taking the necessary depth, and it was made sufficiently wide to admit of painting.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

TRANSACTIONS

Paper No. 1231

THE LARAMIE-POUDRE TUNNEL.*

By Burgis G. Coy, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. W. C. Hammatt, W. L. Saunders, Lazarus White, O. J. Swensson, B. H. Wait, C. Raymond Hulsart, and Burgis G. Coy.

The Laramie-Poudre Tunnel, one of the most important features of the irrigation system now being constructed by the Laramie-Poudre Reservoirs and Irrigation Company for the Greeley-Poudre Irrigation District, has recently been completed, and the remainder of the system is advanced so far that water can probably be furnished for the lands in the district during the season of 1912.

The Greeley-Poudre Irrigation District comprises an area of 125 000 acres of arid land in Weld County, Colorado, in the Valley of the Cache La Poudre River. This land is adjacent to but above the present irrigated area of the Cache La Poudre Valley, and, when supplied with water, will be as productive as any now under cultivation. As the normal flow of the Cache La Poudre River had been already appropriated, it was necessary to look to other drainage areas for water before the present cultivated area could be extended.

The initial steps which led to the development of the Greeley-Poudre Irrigation System were taken in 1902, although it was not then contemplated that this district should ever come into existence. The season of 1902 being a dry one, many of the farmers under the exist-

* Presented at the meeting of April 17th, 1912.

ing ditches found themselves short of water, and began to look around for an additional supply. Messrs. Wallis Link and A. I. Akin, of Fort Collins, Colo., being familiar with the head-waters of the Laramie River, started an investigation to determine whether or not they could divert a part of the waters of that stream into the Cache La Poudre River to supply their crops in time of shortage. It remained for them and their associates to lay the foundation of what has been developed and perfected as the Greeley-Poudre System, which will require the expenditure of approximately \$5 000 000 for its construction.

The first plan was to divert some of the tributaries on the west side of the Laramie River, through a ditch, at an elevation about 10 500 ft., into the West Fork of the Laramie, above the head-gate of the Sky Line Ditch, which had already been built and was being operated by the Water Supply and Storage Company, diverting water from the West Fork of the Laramie through a low pass into Chambers Lake, the head of the Cache La Poudre River. The Sky Line Ditch is shown on Fig. 4. An important part of this plan was the development of a number of small natural lakes, known as the Link Lakes, on the heads of these tributaries, into storage reservoirs, and the operating company was known as the Link Lakes Company, and later, as the Laramie-Reservoirs and Irrigation Company. Further investigation, however, proved that by a tunnel, at an elevation of approximately 8 600 ft., through Green Ridge, the divide between the water-sheds of the Laramie and Cache La Poudre Rivers, a much larger supply of water could be developed.

In the spring of 1907 the Laramie-Reservoirs and Irrigation Company was consolidated with the Mitchell Lakes Reservoirs Company and the Eastman Canal and Reservoir Company, two small reservoir companies operating in the Poudre water-shed, forming the Laramie-Poudre Reservoirs and Irrigation Company; and the tract of land later formed into the Greeley-Poudre District was selected as the most feasible for its development.

The Greeley-Poudre system is naturally divided into two parts, the mountain or collection system, and the plains or distributing system. The principal features of the mountain division are the east and west side collection ditches, 8 and 4½ miles long, respectively, on either side of the Laramie River (Fig. 1). These intercept the flow of the numerous tributaries and divert it back to a reservoir known as

Tunnel Reservoir, which lies in the bed of the river and from which the water is diverted by the Laramie-Poudre Tunnel through Green Ridge into the Cache La Poudre River. Each of these collection ditches has a capacity of 275 cu. ft. per sec., where it discharges into the reservoir, and the capacity from there to the head is decreased in proportion to the water collected.

The cross-section of the tunnel is 7½ ft. high and 9½ ft. wide; its slope is 1.7%, and its capacity is 800 cu. ft. per sec. As it was anticipated that the corners would not break out square without extra drilling work, the dimensions were made large enough to give a minimum section of 62 sq. ft., and still leave quite an area in the corners, the cross-section thus approaching nearer an ellipse than a rectangle of the given dimensions. As actually constructed, however, the section is nearly rectangular, thus giving a section considerably larger than the minimum required, with a corresponding increase in capacity.

The system receives the drainage from the east slope of the Medicine Bow Range (Fig. 2) and the west slope of Green Ridge, located in Townships 7, 8, 9, and 10 North, Ranges 9 and 10 West of the 6th Principal Meridian, and at elevations ranging from 8 600 to more than 14 000 ft.

The distributing system includes the main canal, diverting the water from the Cache La Poudre River, and the distributing laterals and several reservoirs. The aggregate length of canal and laterals is 300 miles, and the storage capacity of all the reservoirs of the system aggregates 100 000 acre-ft.

In September, 1909, the final location of the tunnel was begun by a party of five men under the writer's direction. A base line, approximately 2 600 ft. long, was laid out in a level place in the Laramie River Valley, and a triangulation system was established from which the length and bearing of the tunnel were determined. Levels were run over the hill and bench-marks were placed at each 100 ft. in elevation; these were subsequently checked by another man. The top of the hill is practically 1 000 ft. above the Laramie River and 1 500 ft. above the Poudre River. A dense growth of jack pine and quaking asp covers the top of the hill, and a great deal of cutting was necessary in establishing the triangulation system and the tunnel line. The final line as first located required eight set-ups of the transit to

FIG. 1.—LARAMIE RIVER VALLEY, LOOKING SOUTH TOWARD THE TUNNEL.

FIG. 2.—MEDICINE BOW RANGE, FROM DEADMAN HILL.

get over the hill (Fig. 3), but later this was reduced to six when the permanent points were set.

The contract for the construction of the tunnel was let to Mr. J. A. McIlwee, of Cripple Creek, who had made a record on the Cripple Creek drainage tunnel. The Company agreed to put up and furnish the camps at each end, build a power-plant, and furnish rails, pipes, cars, etc., and the contractor was to furnish his own drills, steel, tools, etc. Mr. McIlwee moved to the site of the east portal on November 25th, 1909, established a temporary camp in tents, and erected a small steam compressor and boiler to use until the Company could erect the permanent camps and power-plant. Although the weather was extremely cold and the ground covered with snow most

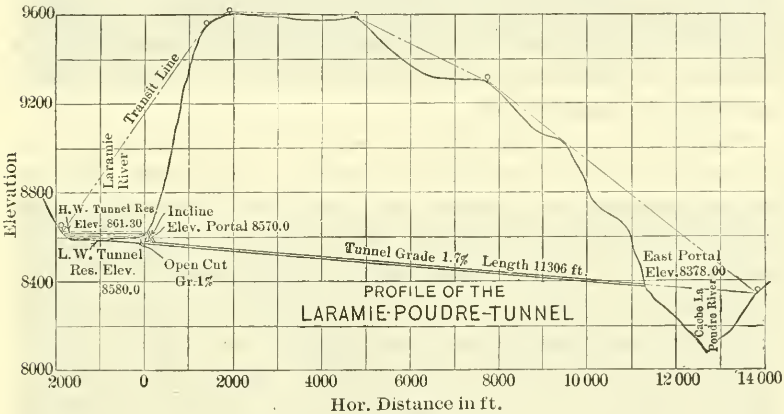


Fig. 3.

of the time, at the end of 30 days he was ready to work, and on Christmas Day the first blow in the actual driving of the tunnel was struck at the east portal. A similar temporary camp and power-plant was erected at the west portal, but, being less accessible, especially at that season of the year, work was not started until nearly a month later. At the east portal 800 ft. of tunnel were driven with this temporary plant, but at the west portal not much tunnel proper was driven before the permanent plant was ready, most of the time having been spent in driving an incline to get down to the tunnel grade.

The contract for installing the power-plant was awarded to the Hampson Fielding Engineering Company, of Denver, Colo., which began operations about December 1st, 1909, and had the plant completed and ready for operation on March 15th, 1910. All the buildings at both ends were erected by the Company by force account.

All the supplies, machinery, and building material had to be freighted in from Fort Collins, Colo., to the east end, and from Laramie City, Wyo., to the west end. The distance from Fort Collins to the east portal is about 65 miles, and the distance is about the same from Laramie City to the west portal. The handling of all this material over mountain roads for such distances, and in the dead of winter, was no small accomplishment. Some of the road was newly built, and was very rough. All traffic to the east portal had to go down Pingree Hill, where a descent of 1 260 ft. is made in 2.7 miles, with several pitches steeper than 20%, and usually covered with ice and snow. Although as great a load as 11 000 lb. was hauled on a wagon, there was no serious accident, and but small loss of material. The price paid for freighting was $1.40 per 100 lb. for the heavy machinery and $1.12½ for lighter stuff.

The Poudre River, near the east portal of the tunnel, has a very steep grade, and affords an excellent site for a power-plant. This was taken advantage of for power for the main plant to drive the tunnel. A 10-ft. rock-filled crib dam was built across the river (Fig. 6) about 1½ miles above the tunnel site, and from this a 22-in. wooden stave pipe leads down the river for 8 500 ft. to the power-plant, where three Pelton wheels are operated under a static head of 278 ft.

A 48-in. single-nozzle wheel, with a maximum capacity of 130 h.p., running at 245 rev. per min., was used to drive an air compressor for operating the drills at the east end, being belted to a 72 by 16½-in. face pulley from a 40 by 16½-in. face driving pulley. The nozzle on this wheel was controlled by the pressure in the air receiver. The compressor was of the Ingersoll-Rand, Imperial, Type 10, cross-compound, 17 by 10 by 14 in., having a capacity of 600 cu. ft. of free air per min., at 135 rev. per min., maintaining a pressure of 135 lb. per sq. in. at the receiver. It was also equipped with an automatic unloading device controlled by the pressure of the air. A 10 by 3-ft. receiver was placed just outside the power-house, and a similar receiver was placed inside the tunnel. The air line for the first 4 000 ft. from the receiver was 4 in. in diameter, then 3 in. for 3 000 ft., and 2 in. for the remainder of the distance; it was reduced at the manifold to the ¾-in. air pipes for the drills.

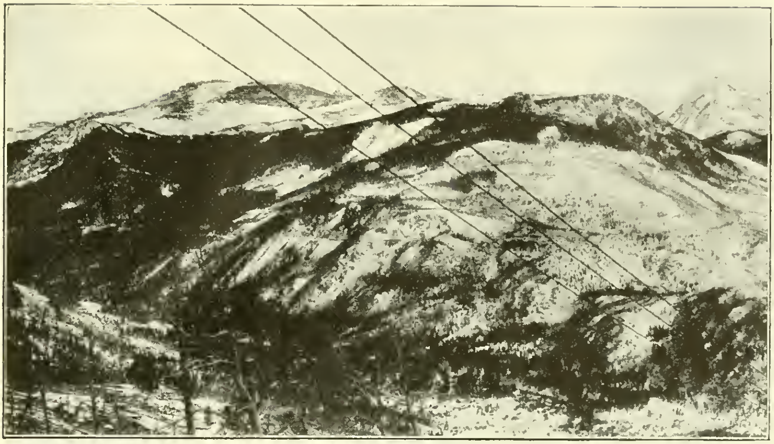


FIG. 4.—WEST FORK OF LARAMIE RIVER, SHOWING SKY LINE DITCH.

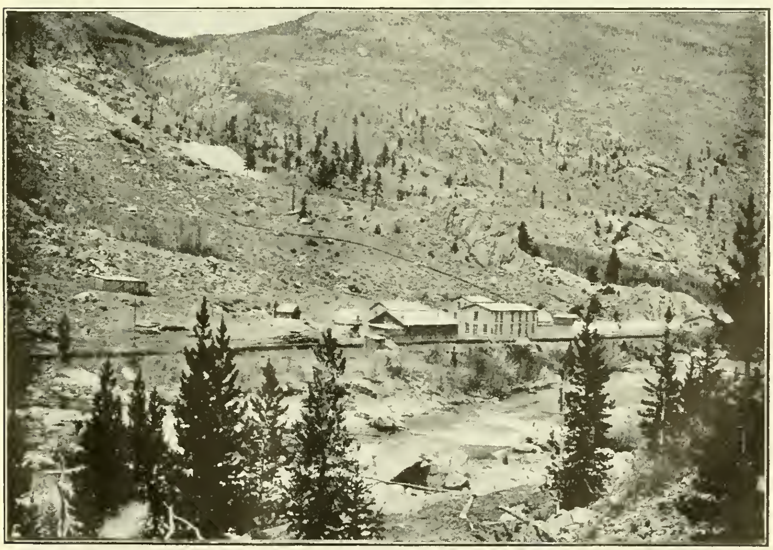


FIG. 5.—EAST PORTAL CAMP ON CACHE LA POUDRE RIVER.

FIG. 6.—DAM AND PIPE LINE FOR POWER-PLANT ON POUDRE RIVER, EAST PORTAL.

A Connorsville blower, with a capacity of 13 cu. ft. per rev., running at 225 rev. per min., was used for ventilating the tunnel, and required from 20 to 30 min. to suck the gas out after each round was shot. This blower was operated by a 48-in. single-nozzle wheel, mounted on the blower shaft, and guaranteed to develop not less than 25 h.p. A 15-in. ventilating pipe was laid from the blower into the tunnel and within 100 ft. of the breast, and was extended as the work progressed.

For lighting purposes in both camps, and for power for the west end, a 150-kw., General Electric, 3-phase, 60-cycle, 2 300-volt generator, running at 600 rev. per min., was used. This was belted to a 48-in. water-wheel, running at 245 rev. per min., operated by a 12-in. double nozzle. There were four nozzle tips, two bored for 125 h.p. each, one for 90 and one for 80 h.p., so that any two could be used in combination as desired, giving an efficiency of 79% when developing 210 h.p., normal load, and 75% when developing 250 h.p. The driving pulley on the water-wheel was 80 by 19-in. face; the driven pulley of the generator was 32 by 19-in. face. A special oil-pressure governor controlled the speed of this wheel.

The equipment at the west end was the same as that at the east end, with the exception of the operating power, all machinery being run by motors, and, for the purpose of hoisting muck out of the tunnel, an F. M. Davis, 25-h.p., electric hoist, was used, having a capacity of 5 000 lb. at a speed of 120 ft. per min. The electricity for operating the plant at the west end was generated at the plant at the east end. The transmission line was of No. 0, weather-proofed, copper wire, 15 000 ft. long, and reached an altitude of 9 600 ft. in crossing the mountains. At each end a 5-kw. transformer furnished current at 110 volts for lighting the camp; and, at the west end, three 150-kw. transformers furnished current at 440 volts for operating the motors.

At both ends of the tunnel No. 7 Leyner water drills were used up to July, 1910, when No. 8 drills were substituted. For the purpose of supplying water for these drills, a $\frac{3}{4}$-in. pipe was carried into the tunnel. At the east end the supply was obtained by gravity from a small creek which runs near the portal. The water was run into a steel tank connected with the compressed air line. When the tank was full the inlet valve was closed and the compressed air was turned into the tank, forcing the water through the pipe to the drills. Two tanks were in use at each end, and one was being filled while the other was being emptied. With this system, freezing caused more or less trouble, but, before the winter of 1910-11 set in, the heading was advanced so far that the tanks could be moved into the tunnel, and the supply was taken from water running on the floor. At the west end the same system was used, except that water was pumped from the Laramie River into a wooden tank on the hillside, above the portal, until the work was advanced so far that steel tanks could be used inside the tunnel.

A No. 2 Leyner drill sharpener was used in the blacksmith shop at each end for sharpening and shanking the drill steel. At the east end the camp buildings (Fig. 5) included: A combination cook-house and dining-room, 28 by 80 ft., large enough for 75 men; a two-story bunk-house, 28 by 80 ft., containing twenty-two bedrooms, accommodating two men each, with a general sitting-room, bath and wash-rooms; a commissary and hospital, 22 by 38 ft., containing storeroom, medicine-room, doctor's bedroom, patients' ward, and bathrooms; a two-story office building, 26 by 32 ft., containing five bedrooms, sitting-room and office; a power-house, 40 by 45 ft., with a small room for the engineer, and a storeroom for supplies; a blacksmith shop, 24 by 24 ft.; a storehouse, 24 by 16 ft.; several 12 by 14-ft. house tents for men who desired to have their families with them; and the usual powder-houses, thaw-houses, barns, and outbuildings. The buildings are all of lumber, and are covered with ruberoid roofing, and the kitchen, bunk-house, and hospital are supplied with running water and sewer connections. The camp at the west portal (Fig. 7) is similar to that at the east portal, except that the buildings are not of the same sizes, and they are of logs, and have ruberoid roofs. There is also a transformer-house, in addition to those already mentioned.

As the grade of the tunnel is to the east, conditions were much more favorable for rapid work at the east end than at the west end, where it was necessary to haul all muck up hill, as well as to pump out the water encountered. The grade of the tunnel at the foot of the hill at the west end is 15 ft. below the bed of the Laramie River, and about 1 200 ft. distant from it. It was thought best to start the tunnel (Fig. 3) at the hill, where the formation was solid and water-

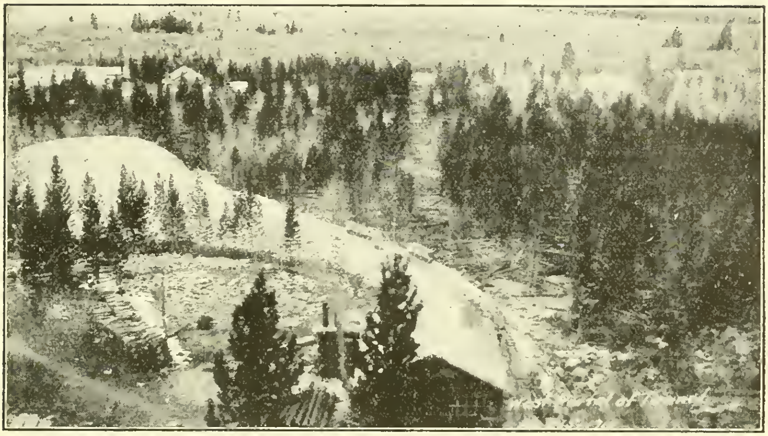


Fig. 7.—West Portal Camp and Dump.

Fig. 8.—Drills on Bar, at East Portal Heading.

tight, and to finish the part back toward the river after the two headings had met. This part, being glacial deposits of sand and gravel, contained more or less water. The tunnel grade is about 40 ft. below the surface at this point, and was reached by an incline about 200 ft. long, starting where the first rock formation showed at the bottom of the hill. After the two headings met, a tunnel was driven back from the foot of the incline toward the river as far as the formation was solid; the remainder was taken out in open cut.

As it was anticipated from the first that more or less water would be encountered at the west end, facilities for handling 700 gal. per min. were provided. A Byron Jackson, two-stage, 5-in. centrifugal pump, direct-connected to a 25-h.p., 3-phase, 440-volt, induction motor, running at 900 rev. per min., was mounted on a truck and kept on a side track at the portal ready to be taken down into the tunnel whenever needed. A 5-in. spiral-riveted water pipe was laid as the tunnel advanced, to which the pump could be connected. It was never found necessary, however, to run this pump, but, for a good part of the time, a small piston pump was in use. This was operated by compressed air from the main air line. A sump and a pump station were blasted out at the side of the tunnel, so that the pump would be out of the way and safe from the flying muck, and the pump was set up and connected to the water line. When the sump filled up, the pump was operated until it was empty again. As the tunnel progressed, new sumps were blasted out, and the pump was moved nearer the breast. At the east end all the water flowed out by gravity.

All the drilling was done with Leyner drills mounted on a horizontal bar, which was held in position by tightening it against the walls of the tunnel with a jack-screw, with which one end was fitted (Fig. 8). The bar was made of a piece of 3-in., double-strength pipe, with a fixed shoe on one end and a screw on the other, and was easily handled and set up by the machine men and their helpers. Some of the holes were drilled with the machines above the bar and some with them below it. Two set-ups of the bar were required to drill the entire round. The upper set-up was drilled from the top of the muck pile while the muck was being cleared away; then the bar was lowered and the lifters were put in. At first two drills were used on the bar, but later it was found that three could be used to much greater advantage. Rounds were drilled from 10 ft.

deep with 12-ft. cut holes, breaking a 10-ft. round, down to 7 ft. deep with 8-ft. cut holes, breaking a 7-ft. round.

Fig. 9 is a diagram of the holes as drilled for a 10-ft. round, by three drills on the bar, drilling 21 holes to the round. The holes lettered *R. T.* were drilled by the right-hand machine on the top set-up. Those marked *C. T.* were drilled by the center machine on the top set-up, and those marked *L. T.* by the left-hand machine; likewise, those marked *R. B., C. B.,* and *L. B.* were drilled by the respective machines on the bottom set-up. The holes were started 2⅛ in. in

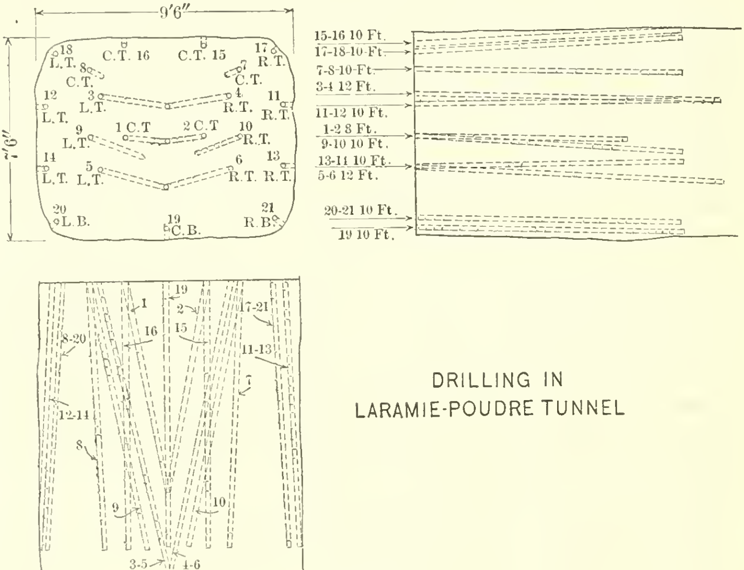


Fig. 9.

diameter and bottomed at 1⅝ in. It will be noted that each man had but one hole to drill on the lower set-up, and when one man was delayed in finishing his lifter, the others blew out the holes already drilled and loaded them ready to shoot.

The holes were fired in the order numbered in Fig. 9, each being fired independently of the others, by fuses cut to such lengths as would explode them in the proper order. The pairs of holes, 1 and 2, 3 and 4, and 5 and 6, however, being joined or very close together, usually went at the same time, and gave better results when they did so.

The charge varied, both as to the strength and quantity of powder used, according to the depth of the holes and the quality of the rock, but a charge for a 10-ft. round in hard rock was about as follows: Seven sticks of 1¼ by 8-in., 100% blasting gelatin were put in the bottom of each of the cut holes and tamped, to within 2½ ft. of the collar of the hole, with 60% dynamite; the remaining holes were similarly loaded with 60 and 50% dynamite. The wrappers were slit and the powder tamped until it filled the hole. German insoloid fuse was used, and 5 x California caps. Hole 21, being the last one fired, threw the muck away from that side of the tunnel, leaving room to operate the lever to tighten the screw in setting up the bar for the next round. Rounds were fired as soon as ready; and, as soon as the smoke was cleared out, usually from 15 to 30 min., the drillers set up the bar for a new round, and the muckers began loading the muck shot down.

The muck was loaded into cars by from four to six men using square-pointed shovels. Steel plates, ⅜ in. thick, were laid on the floor of the tunnel, extending back about 25 ft. from the breast, and were covered with enough muck to keep them in place while shooting. The muck fell on these plates and was easily shoveled from them. Steel cars, of 18 cu. ft. capacity, were used on a single track. A trip of empty cars was run up close to the muck pile, and all but one were then tipped off the track to one side; the remaining car was loaded and pushed back past the empty cars, one of which was then put on the track and loaded in the same way. When all the cars in the trip were loaded they were pushed up as close to the muck pile as possible and an empty trip was run up close to them and tipped off the track; then the loaded trip was run out to the dump, and a new trip was loaded as before.

At the east end the loaded cars ran out by gravity, and were hauled back by mules. At first one mule, handling a 5-car trip, was able to keep the muckers busy, but, as the haul increased, two mules in tandem were used on 10-car trips (Fig. 10). Two mules in this way were able to handle the muck until the tunnel was in about a mile, and there a siding, long enough to hold about 40 cars, was laid. At the siding the two mules left their empty trip and returned with a loaded one, another mule hauling the empty cars from the switch to the heading in 5-car trips and returning to the switch with a loaded trip. As the length beyond the siding increased, two mules and 10-car trips were substituted. The mules took the cars to the dump track, left them for the dumpman to unload and arrange on another siding ready to go in, and returned with a trip already on this siding (Fig. 11).

At the west end a mule hauled the empty cars and the hoisting cable to the breast and did the shifting there, but loaded cars were hauled out by the hoist, taking trips of 12 cars to the foot of the incline and 3 or 4 cars up the incline, where a man with another mule took them to the dump and returned the empty cars to the siding ready to go in again.

Dull steel was brought out on the loaded trip, left at the blacksmith shop as the trip went by, and sharp steel was taken in with the empty trip.

The track, with 18-in. gauge and 16-lb. rails, was laid on the left side of the tunnel; the ventilating pipe, air and water lines were laid on the right side. As the ventilating pipe did not extend nearer the breast than 100 ft., this gave room to handle the cars.

The rock encountered was mostly a hard gray or red granite. Some soft seams were encountered, however, and where these ran in the direction of the tunnel, some timbering was necessary. In all, eighteen different stretches have been timbered, varying from 15 to 400 ft. in length, and aggregating 985 ft. In most cases, however, the rock stood until the tunnel heading was far enough past to allow the timbering to be put in without interfering with the progress to any great extent. The timbering was of the ordinary square-set type, made from round logs and lagged with poles, all of which were cut from the hillsides near by. The pieces in the square sets varied from 9 to 18 in. in diameter, and the spacing was from 3 to 8 ft., according to the quality of the rock supported. The timber used was fire-deadened red spruce, with an occasional stick of white spruce or lodge pole (Fig. 12). It is the intention to line all these timbered places with concrete during the winter of 1911-12.

The drillers, helpers, and muckers worked in three 8-hour shifts of 3 drillers, 2 helpers, from 4 to 6 muckers, and 1 foreman. Drivers, blacksmiths, and power-house men worked 12 hours. The wages paid were as follows: Drill runners $4.50, helpers $4.00, muckers $3.50,

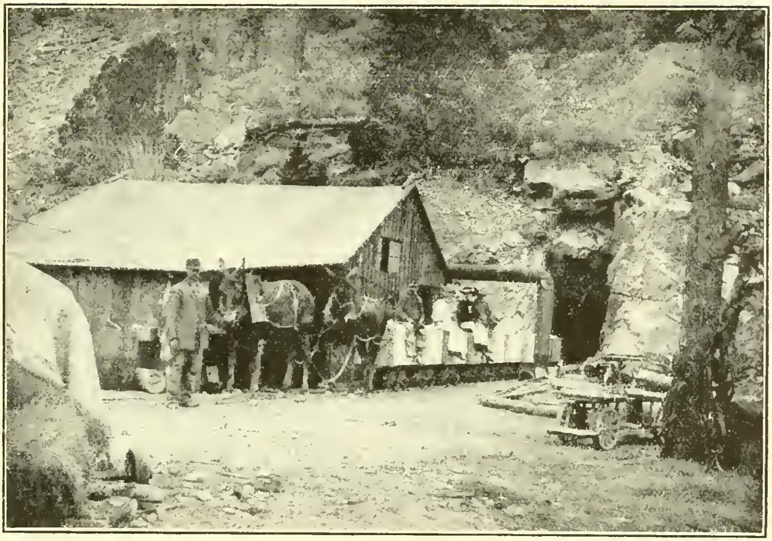


FIG. 10.—TRIP OF CARS AT EAST PORTAL OF TUNNEL.

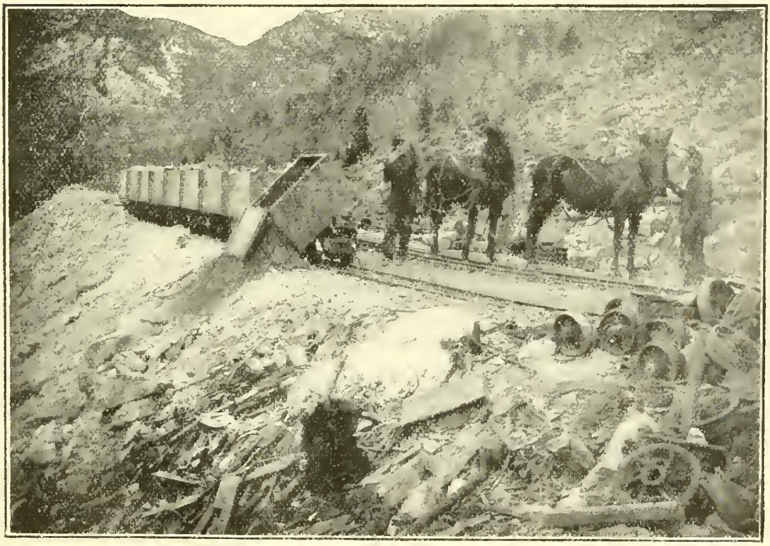


FIG. 11.—DUMPING CARS AT EAST PORTAL OF TUNNEL.

blacksmiths $4.50, drivers $4.50, and foremen $6.00. In addition to these wages, a liberal bonus was given each month for fast work.

The total length of tunnel driven is 11 306 ft., and the progress for each month was as follows:

January, 1910, east end	302		
February, "	315		
March, "	350	West end	202
April, "	354	"	279
May, "	513	"	336
June, "	429	"	388
July, "	443	"	371
August, "	527	"	293
September, "	485	"	292
October, "	413	"	28*a*
November, "	424		
December, "	482		
January, 1911	609		
February, "	420		
March, "	653		
April, "	583		
May, "	635		
June, "	576		
July, "	416*b*	West end	81*c*
August, "	...	"	107*d*

This gives a monthly average of 308.7 ft. for 7 months at the west end, and 473.7 ft. for 19 full months at the east end; 509.4 ft. per month for the 16 months during which the complete plant was operated, and 525.2 ft. per month for the last year of work; 653 ft. in March, 1911, sets a new record in America for 1 month's work.

This remarkable record was due to the organization of the operating force, the friendly rivalry between the different shifts, and in no small measure to the efficiency of the power-plant, which was operated for more than 16 months. The total time lost from all causes aggregated less than 24 hours.

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a. Work was shut down at west end on October 4th.

b. Headings met July 24th.

c. Work at west end under incline, but handled from east end.

d. Work at west end under incline, completed August 8th.

On checking up, after the connection was made, the following errors were found: alignment 0.01 ft.; grade 0.18 ft.; computed length between initial points 11 288.77 ft., measured length 11 288.20 ft., or an error of 0.57 ft. For the alignment, three separate lines were run over the hill from the west to the east, and the mean of these was run back through the tunnel for a center line of the east end work. Three different sets of levels were carried over the hill from the west to the east, and the mean of these was assumed as the correct elevation from which to carry the elevation back through the tunnel. The measured distance through the tunnel is the mean of two measurements. The point at which the headings met (Fig. 13) was 8 937 ft. from the east portal and 2 351 ft. from the top of the incline at the west portal.

Mr. J. J. McIlwee, son of the contractor, was Superintendent at the east end, and Mr. Walter Warner at the west end. Mr. D. W. Brunton, Past-President of the American Institute of Mining Engineers, was Consulting Engineer. Charles R. Hedke, M. Am. Soc. C. E., of Fort Collins, was Chief Engineer until June, 1911, when he was superseded by Mr. L. L. Stimson, of Greeley, Colo. The writer was Resident Engineer in charge of the tunnel, collection ditches, and reservoirs.

From the end of the solid formation on the west side, an open cut extends to the river, a distance of 1 500 ft. The total quantity to be moved is about 35 000 cu. yd., the cuts ranging from 0 to 44 ft. This work is now under contract by Messrs. Iauson and Loesch, of Fort Collins. The first 300 ft. next to the hill consists of gravel and boulders; many of the latter contain from 10 to 20 cu. yd., and require considerable blasting. This part of the work was sub-let to a station gang of Swedes who are doing the work by hand, using a track and cars drawn by a horse to handle the muck. The remainder of this cut is of gravel and clay, and is being done by teams in the usual way.

The controlling works at the west end of the tunnel consist of a set of three steel gates, each with a 5 by 8-ft. opening. They are set in concrete in the usual way for river gates. It was originally planned to have a 50-ft. dam at the tunnel reservoir, giving a draw of 33 ft. through the tunnel, but for the present the dam will be only

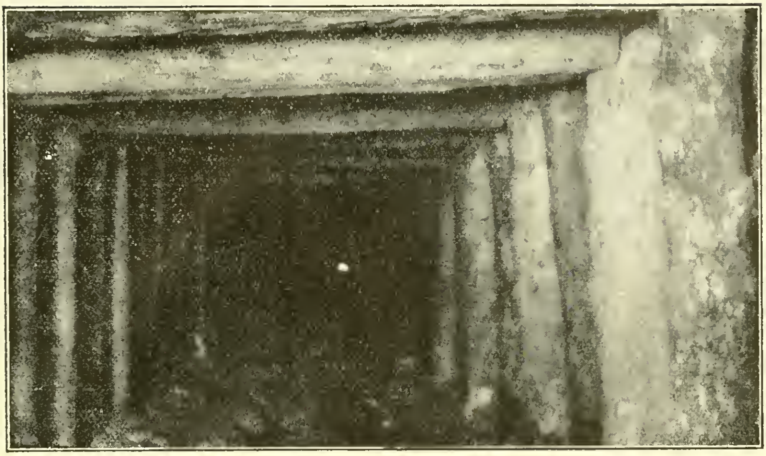


FIG. 12.—SECTION OF TIMBERING IN TUNNEL.

FIG. 13.—INTERIOR OF TUNNEL AT JUNCTION OF HEADINGS.

26 ft. high, giving a draw of 9 ft., and the gates are put in the cut, near the high-water line.

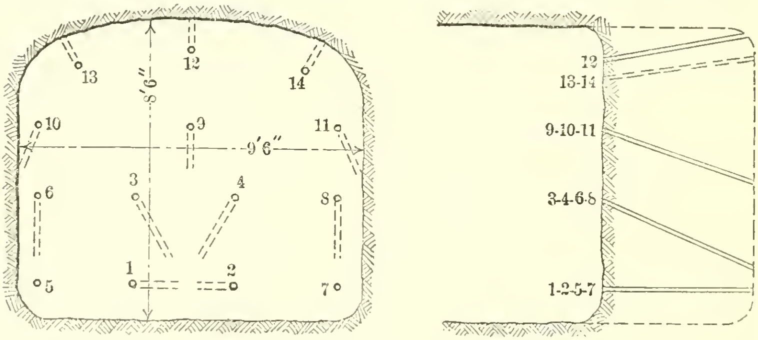
At the east end of the tunnel the water will be discharged into a concrete box, 30 ft. square and 14 ft. deep. A concrete flume, 30 ft. wide and 50 ft. long, will draw the water from this box, leaving the lower 6 ft. of the box for a water cushion. After rating, the water will be released to run down the hillside to the Cache La Poudre River, about 1 200 ft. away, and 300 ft. below the end of the tunnel.

DISCUSSION

Mr. Hammatt.

W. C. Hammatt, M. Am. Soc. C. E. (by letter).—The writer would like to have additional data in regard to the detailed cost of the Laramie-Poudre tunnel. The author neglects to give either the contract price per unit, or the estimated cost to the contractor.

In 1905 the writer had charge of driving a working tunnel for reaching an ore body in Shasta County, California. This tunnel was of approximately the same cross-section as the Laramie-Poudre Tunnel, and complete cost data were kept. The tunnel was 9 ft. 6 in. in width and 8 ft. 6 in. in center height, and was mostly in hard felsite, although partly in felsitic porphyry, which part amounted to less than one-third and was timbered. Burleigh drills, of the Company's own manufacture, were used, with solid-cross steel bits, the power for operating them being obtained from a central compressor plant receiving power from the Northern California Power Company at a



DRILLING DIAGRAM, 14-HOLE ROUND.
Cut holes, 5'6"; other holes, 5'0".

Fig. 14.

cost of 1 cent per kw-hr. As there was no cause for rushing the work, only a single drill crew operated in the heading, although there was ample space for two without interference. The blacksmithing was done in the general blacksmith shop, which handled much other work, but only its proportion was charged against the tunnel. This was also true of power-house expenses and superintendence. The mine had a narrow-gauge railroad, which made transportation charges light.

It will be noted from the drilling diagram, Fig. 14, that a round consisted of fourteeen holes, and that a shot contemplated the breaking of 5 ft. of ground. The results are shown in Table 1, and the writer is interested to know the relative advantages of the 21-hole round and the 10-ft. advance. The author's experience with the Leyner water drill, as to speed of drilling and cost, would also be of interest for comparison with the solid-bit type. The writer is aware that the former finds great favor in Colorado, though little used in California. Mr. Hammatt.

TABLE 1.—Detailed Cost of Lower Fielding Tunnel, Iron Mountain, California, 1905.

	Labor.	Supplies.				Tool sharpening.	Track laying.	Power.	Interest and depreciation.	Superintendence.	Total.
		Explosives.	Timber.	Tools and sundries.	Total.						
Drilling....	$1.57	....	....	$0.286	$0.286	$0.086	....	$0.281	....	....	$2.223
Blasting...	0.504	$1.047	....	0.014	1.061	....	....	....	....	....	1.565
Mucking...	2.36	....	....	0.22	0.22	0.04	....	0.094	....	....	2.714
Tramming.	0.90	....	....	....	....	....	$0.349	....	....	....	1.249
Timbering.	0.84	....	$0.676	....	0.676	....	....	....	....	....	1.516
General expenses.	....	....	....	....	....	....	....	....	$0.851	$0.334	1.185
Totals....	$6.174	$1.047	$0.676	$0.520	$2.243	$0.126	$0.349	$0.375	$0.851	$0.334	$10.452

Cost of drilling, per foot drilled...............	$0.151
Cost of timbering, per foot timbered...........	$4.38
Cost of timbering, per set......................	$27.74
Dynamite used per foot of tunnel...............	7.88 lb.
Feet drilled per drill sharpened................	3.5
Feet drilled per foot of tunnel.................	14.75
Cost per cubic yard..........................	$4.04

W. L. Saunders, M. Am. Soc. C. E. (by letter).—Mr. Coy has rendered a service to engineers in his admirable paper, which calls attention to a small but important piece of tunnel work recently completed. The Laramie-Poudre Tunnel, under ordinary circumstances, might attract but little attention. It is short, as tunnels go nowadays, of moderate section, its construction involved no special difficulties, and in many respects it resembles dozens of tunnels that have been built throughout the world, especially in America. Mr. Saunders.

There is one great difference, however, between the Laramie-Poudre Tunnel and any other. Unusual speed of driving has been attained without the use of a carriage and by a system which may be called a modified American system of tunnel driving. To begin with, it must be understood that there is a distinct difference between European and American systems. The great tunnels of Europe—the Mont Cenis, Saint Gothard, Arlberg, Simplon, and Loetschberg—are the greatest tunnels in the world. These tunnels, especially the

Mr. Saunders. Simplon and Loetschberg, have been driven by a system which has been perfected through long experience to such a degree that, at least as far as speed of driving is concerned, nothing has equalled it.

The Simplon Tunnel.

The Simplon is the longest railway tunnel in the world, the distance between portals being 12¼ miles. The elevation of the Swiss portal is 2 250 ft. and of the Italian portal 2 076 ft. above sea-level; the highest point in the tunnel is midway between the portals, and is at an elevation of 2 310 ft.; from this summit-level the line descends on a 2% grade to Brigue, and on a 7% grade to Iselle at the Italian end.

The work consists of twin single-track tunnels, exactly parallel in plan and profile, and lined throughout with masonry. The centers of the tunnels are 55.76 ft. apart; at the summit-level the cross-section is increased in dimensions to accommodate two tracks.

A center bottom drift was first driven by power-drills, and then timbered and covered with a closely-boarded roof. From this drift a shaft was driven upward to the roof-line every 164 ft. (50 m.). The top heading was then excavated by working in both directions from each of these shafts. Next in order, the floor of the upper heading was removed and then the two side cheeks of the bottom drift. The lower drift being timbered, no interruption of the traffic in it was caused by the removal of the rock above.

Drilling.—The advance drift was the only part of the operation performed by power-drills. The drills used were Brandt rotary machines mounted in groups of two on a heavy thrust-bar about 12 in. in diameter. This thrust-bar was pivoted to a drill-carriage and counterbalanced.

The section at the heading was nominally 6.5 by 9.5 ft., or 61.75 sq. ft., and as the depth of each blast was roughly 4.5 ft., the material removed by each blast ranged from 265 to 275 cu. ft.

The average daily advance was about 16 ft. at the Italian end and from 20 to 21 ft. at the Swiss end. This work was in gneiss. In rock of more friable nature, such as anhydrite or calcium sulphate, an advance of as much as 34 ft. in 24 hours was made. After each blast, the time required to clean the heading, set the drills, complete the boring, and remove the drill-carriage, was more than an hour.

Explosives.—The explosives used were dynamite at the Italian end, and blasting gelatine at the Swiss end. The dynamite was put up in packages weighing about 1 lb.; each hole was charged with six cartridges; each blast in the drift, therefore, used from 60 to 66 lb. of powder, or about 6.5 lb. per cu. yd. Charges were fired by ordinary fuses, cut so as to give an interval between the firing of successive holes; about 15 min. was required after each blast to clear the fumes from the heading. This was accomplished with a ventilating pipe running close to the face and the use of a spray of water. The ventilating pipe exhausted about 35 cu. ft. of air per sec., and the spray absorbed the sulphurous gases. Mr. Saunders.

Mucking.—The spoil was cleared from the face by one gang while another gang loaded the collected muck into narrow-gauge cars hauled by horses. No machines were used, all the material being handled by manual labor. The work of clearing the heading was rushed to enable the drills to be put to work as soon as possible. To this end the clearing gangs were composed of men who had been previously rested by performing light work, and only the most skilled and energetic laborers were employed. The majority of the workers were from Southern Italy. There were 14 or 15 men at each heading, working in three shifts daily. Each gang had two horses for each shift. Horses, which cost $1.60 per 8-hour shift, died off rapidly, and were paid for by the tunnel contractors. Other methods of transportation were tried, but proved less economical than horses in the advance headings. The horses took the cars to the compressed-air locomotives, and these in turn took them to the steam locomotives.

The time taken for each portion of the attack was as follows:

Bringing up and adjusting drills	20 min.
Drilling	1 hr. 45 min., to 2 hr. 30 min.
Charging and firing	15 min.
Clearing away débris	2 hr.

One whole attack required from 4.5 to 5.5 hours, resulting in an advance of 3 ft. 9 in., or a daily advance of 18 ft. From this it appears that the time spent in clearing away the spoil equalled that consumed in drilling, and it is in this clearing that a saving of time is likely to be effected, rather than in the process of drilling.

The average temperature at the face was 73° Fahr. during drilling operations, 76° Fahr. after firing, and a maximum of 80° Fahr. on the south side, with 80° Fahr. and 85° Fahr. before and after firing.

Rate of Progress.—The progress for three months is given in the trial report for 1900, as follows:

"At Brigue, where there were three drilling machines in one heading and two in the parallel heading, the total length excavated was 995 yd., or 6 409 cu. yd., in 89 working days. The average cross-sectional area was 57 sq. ft. This required 507 attacks and 3 066 holes, which had a total depth of 26 600 ft., and 14 700 re-sharpenings of the drilling tool.

"At Brigue 648 men and 29 horses were employed at one time in the tunnel. At Iselle the numbers were 496 men and 16 horses, work-

Mr. Saunders. ing in shifts of 8 hours. Outside the tunnel, in the shops, forges, etc., the men work from 8 to 11 hours per day, the total being 541 men at Brigue and 346 men at Iselle.

"On the Italian side, where the rock is very much harder, there were three drilling machines in each heading; the total length excavated, with a cross-sectional area of 62 sq. ft., was 960 yd., or 6 700 cu. yd., in 91 working days. This required 61 293 re-sharpened tools, 758 attacks, 7 940 holes, with a total depth of 33 000 ft., and 56 000 lb. of dynamite. The average time spent in drilling was 2 hours, 55 min., and in charging and clearing, 2 hours, 36 min.

"Thus, in the hard gneiss, to excavate 1 cu. yd. of rock required 8.5 lb. of dynamite and each tool pierced 6.5 in. of rock before it required re-sharpening."

The Loetschberg Tunnel.

The Loetschberg Tunnel, driven through the Bernese Alps, in Switzerland, is the last link of a railroad system connecting the City of Berne directly with the Village of Brigue, which is at the north portal of the Simplon Tunnel. With its completion, and the lately finished 12 000-ft. Weisenstein railroad tunnel located about 30 miles north of Berne, it forms the shortest route between London, Paris, Brussels, or Hamburg, and Genoa, *via* Berne, Thun, Brigue, and Milan.

The question of connecting the Bernese Oberland with the Rhone Valley had its origin as far back as 1866, and the present location of the tunnel was proposed in 1899. In that year two consulting engineers, at the request of the Bernese Government, began a careful study of the location of the proposed road, and reported in favor of a single-track tunnel 44 500 ft. long, basing their estimate on an average cost of $4.90 per cu. yd. for tunnel excavation and $6.55 per cu. yd. for masonry lining throughout the tunnel length. The total cost of the tunnel was estimated at $107 per lin. ft.

Assuming an average progress of 4 ft. per day for hand-drilling, or from 5 to 8 ft. for machine-drilling, and from 15 to 18 ft. for rapid driving, the time required for completing the tunnel was estimated at 5 years. The maximum rock temperature expected was 95° Fahr.

Later, however, the expected increase of the traffic through the Simplon Tunnel brought out the question of accommodating two tracks in the proposed tunnel, and therefore it was decided to drive a double-track tunnel.

Estimates were prepared, and the cost of the new proposed tunnel was calculated to be:

Tunnel excavation and lining	$8 660 000
Tracks, installations, etc.	1 400 000
Total	$10 060 000

or a total cost of $211 per lin. ft.

The main tunnel is 47 678 ft. long, and was first planned to be on a tangent. After the cave-in of July 24th, 1908, it was found necessary to insert a curve of 3 600 ft. radius in the tunnel in order to drive through solid rock. The maximum grade in the tunnel is 7 per cent. The distance from the south portal to the end of the line, at Brigue, is 15.75 miles. Of this length, 28%, or 23 200 ft., consists of 21 tunnels, the longest being 4 450 ft. Of this stretch, 54% is on curves, and 90% on grade. The difference in elevation between the south portal and Brigue is 1 110 ft., and the maximum grade is 2.7 per cent. Mr. Saunders.

Summarizing, the total length of the road is 45.8 miles, of which 36%, or 86 900 ft., is tunnels. Let it be added that, for construction purposes, narrow-gauge railways had first to be built to reach each portal. On the south side of the tunnel the construction railway necessitated 38 tunnels, aggregating 18 000 ft. Of the 38 tunnels, 11 only will be part of the permanent road.

Rate of Progress.—Driving the headings was begun on October 1st, 1906, for a single-track tunnel, and continued until October 1st, 1907, when it was decided to drive a double-track tunnel; 86% of the tunnel had been driven by October 31st, 1910. The headings met on March 31st, 1911. On October 31st, 1910, the 4 000 ft. of heading which had been abandoned after the cave-in of 1908 had been regained.

The power-plant for the south heading, situated at Goppenstein, was driven by electric power. The current was brought at 15 000 volts, and stepped down to 500 volts for power purposes.

Compressed air for the drills (Ingersoll-Rand) was furnished by three 2-stage Ingersoll-Rand compressors, each having a capacity of 1 950 cu. ft. of free air per min., and a compression of 145 lb. per sq. in. They were driven by 400-h.p. electric motors. Compressed air for the locomotives was furnished by two 4-stage Ingersoll-Rand compressors, having a capacity of 460 cu. ft. of free air per min., and a compression of 1 760 lb. per sq. in. They were driven by 250-h.p. electric motors.

The power-plant for the north heading was situated in Kandersteg. Electric power, used throughout the works, was brought from Spiez at 15 000 volts, and stepped down to 500 volts for power purposes in the tunnel as well as in the shops.

Compressed air for the drills (Meyer) was furnished by two units, each consisting of a 2-stage, Meyer, air-compressor, each having a capacity of 1 770 cu. ft. of free air per min., and a pressure of 117 lb. per sq. in. They were belt-driven by 450-h.p. electric motors.

Compressed air for the locomotives was furnished by two units, each consisting of a 5-stage, Meyer, high-pressure compressor, with a capacity of 565 cu. ft. of free air per min., and a pressure of 1 760 lb. per sq. in. They were belt-driven by a 250-h.p. electric motor.

Mr. Saunders.

A drill-carriage of simple but efficient design was devised by the contractors. Each carriage carried four or five drills. It can be easily seen that the drills could be swung independently through an arc of a circle or moved sideways, while they could be given different positions by being swung in a vertical plane.

The time required to change the machine from a position where the drills were not in use to that required for actual drilling was usually from 6 to 8 min. This fact alone shows the superiority of this system of carrying the drills for such work over any other method used up to the present time.

Labor and Wages.—Italian labor, mostly from the northern provinces, was used throughout the works, with the exception of some Macedonians lately imported.

A bonus system of payment was used throughout the different kinds of operations. The following wages were paid:

	Daily wages.	Average bonus.	Total.
Drill-foreman	$1.50	$1.10	$2.60
Drill-runners	1.00	0.70	1.70
Muckers	0.80	0.50	1.30
Nippers	0.70	0.30	1.00
Tracklayers	0.80	0.15	0.95
Masons	1.00	0.40	1.40

There were three 8-hour shifts per day.

The width of the finished tunnel section is 28 ft. at the arch springing and 25 ft. at the base of the rail. The arch is semicircular, the crown being 20.7 ft. above the base of the rail.

As to the sequence of excavation, a bottom heading, 6.5 by 10 ft., was first driven several hundred feet in advance of the enlargement. Upraises, from 500 to 600 ft. apart, were then driven, and a top heading was started back and forth. The top heading was then enlarged.

In the bottom heading the mining operations were as follows: The drill carriage was run forward from its siding close to the face of the heading, passing over 5 by 5-ft. by $\frac{3}{8}$-in. steel plates laid on the floor of the heading for a length of about 30 ft. Each plate had 1-in. holes at the corners for ease in handling with a pick.

The water and air pipes, laid on one side of the heading to about 40 ft. from its face, were connected with the drill carriage, and the drilling began with the top holes. Water sprinkling was done frequently, especially in starting the holes, in order to lay the dust.

Without interfering with drilling, mucking was going on just behind the drill carriage, and the loaded muck-cars were run back to a siding, where trains of from 20 to 30 cars were formed and hauled out by air locomotives.

Mr. Saunders.

Drilling having been completed in the heading, the drill carriage was run back to its siding, and the steel plates laid on the floor were covered with a layer of muck about 4 in. thick, to prevent deterioration. The bore-holes were then loaded and carefully tamped, and the last man to leave the heading, after firing the fuses, opened the air pipe valve, the escaping air thus creating a cushion of fresh air from the face of the heading back to a certain distance, so that, after blasting, the muckers were able to go to work without delay.

In the heading, only a high-grade explosive was used, and this broke the rock into small pieces and rendered mucking with shovels easy. The bore-holes, having an average depth of about 4 ft., were started with 3-in. drills and finished with 2-in. drills. On account of giving better results, firing was done with fuses about 4 ft. long, the center holes being fired first.

Mucking operations were as follows: Two empty cars were run to the heading, the first one being immediately loaded by two or three men shoveling without interruption until the car was fully loaded. This operation was performed in 3 or 4 min., which means that 1 cu. yd. was loaded in from 2.5 to 3 min.

Owing to the manner of drilling and blasting and to the shallow holes, the muck, instead of piling up in front of the face of the heading, was thrown back, and formed a layer over the floor, which enabled the track to be cleared rapidly.

Getting rid of the muck is always a problem in tunnel-driving. At Loetschberg a cubic-meter car (35.5 cu. ft.) was filled in 5 min., and it took only 1 min. to get this car away and bring an empty car to the heading. In order to do this, small entries or chambers were excavated at intervals in the lateral wall of the main heading, which permitted an empty car to be thrown from the track on the side, thus clearing the track and allowing the filled car to pass, whereupon the empty car was turned up on its wheels and rolled into the heading. This is an illustration of an improvised siding in a narrow heading, by which one car may pass another.

Drilling was started not more than 5 min. after the removal of the last car load. This result, which at first sight seems impossible, was only obtained by absolute discipline.

The man who knows that his only work at this moment is to connect the air main to the drill carriage does not do anything else; the men whose duty it is to screw the carriage tightly to the wall immediately jump to the right place.

The system of a low and wide gallery was adopted, the proportion being 1:2, as the gallery is 6 ft. high and 12 ft. wide. The rate of drilling was 15 or 16 holes in from 1.1 to 1.15 hours.

Mr. Saunders.

An engineer who recently visited this work says:

"When I arrived at the heading it was 9.30 A. M. The holes were being prepared for blasting. The blast took place at 9.35 A. M.; 5 min. after the blast the men were in place removing the débris, and at a little after 11 A. M. the drill-carriage was in place again and the rock-drills were working. It usually takes from 25 to 30 min. between the time at which the drilling is finished and the time at which the start is made to remove the débris; that is to say, 25 min. for taking away the drill-carriage, cleaning the holes, loading with explosives and blasting. An additional 5 min. are consumed in getting the smoke away by means of the ventilator, and then the men get to work at the débris. In order to assist the men, a spray of water is discharged near the heading after the blast. This water is brought into the tunnel in a pipe placed within a larger pipe, which insulates it and keeps its temperature from being affected by the temperature of the tunnel."

Drilling in the top heading was accomplished with two or three drills, carried on tripods or on a horizontal bar, while hammer hand-drills were used generally for the enlargement.

Mucking operations in the top heading were very simple, since all blasted material was dumped directly through the upraises into cars running on a siding in the bottom heading.

The operations of blasting, mucking, timbering, and hauling were performed without interruption and without interference with each other, and a special force of engineers was required in order to obtain such a result.

The nature of the rock was different on the two ends of the tunnel. An average of one steel was required in the north end for 1 cu. m. of excavation, while on the south end an average of from 5 to 7 was required for the same work. The average consumption of steels for 2.5 years was: north end, 2.33; south end, 7.70 per cu. m. driven.

The report for 1910 shows that in the first part the rock encountered on both the north and south ends was practically the same, although the average drilling time was much less on the north than on the south end. To explain this, the air pressure on the north end during this period was 7.75, as compared with 5.7 atmospheres on the south end, which largely accounts for the difference. It must also be noted that the number of steels per cubic meter of rock removed was, on the north end, 4.65, and on the south, 8.61, which indicates either that the effect of the rock on the drill-bits was different, or that the blacksmithing work was unequal.

The things most important about these great Alpine tunnels are that in both of them a form of tunnel carriage was used which very materially aided the progress. This carriage is in no sense the type of carriage introduced in the Hoosac Tunnel and attempted in various tunnels, notably the Trans-Andine, the difference being that all the old style carriages, the original Burleigh, for instance, which was one of the first of its type, involved an apparatus which practically filled the entire area of the heading. This never was a success; that is, it never accomplished what its designers sought. The Alpine tunnel carriages are no larger in area than a baby carriage. They consist of a simple little car, mounted on four wheels, combining nothing more than cast iron and steel. This car runs on a narrow-gauge sectional track serving as a means by which a heavy shaft bar may be carried into the heading and jacked in position. The drills were mounted on the shaft bar, and whether in the case of the hydraulic drills used at Simplon or the percussive drills used at Loetschberg, the machines were heavy and could not have been handled without some form of carriage. Mr. Saunders.

With the use of these heavy drills and tunnel carriages the following records have been made:

At the Simplon the average daily advance per heading was about 16 ft. at the Italian end and from 20 to 21 ft. at the Swiss end. This was in gneiss. In less difficult rocks an advance as high as 34 ft. in 24 hours was made.

At Loetschberg yearly records per heading ran as high as 21 and 27.6 ft. per day while monthly averages per heading were as high as 32.3 and 33.9 ft., with some daily averages beyond anything ever achieved in tunnel driving.

At Laramie, Mr. D. W. Brunton, in his paper presented at the San Francisco Meeting of the American Institute of Mining Engineers, in October, 1911, says:

"From March 1st, to March 8th, 1911, inclusive, the tunnel was driven in a single heading, 192 ft., a daily average of 24 ft.; but the highest rate of progress was made during the last four days of January, 1911, when the tunnel was driven 112 ft., or 28 ft. per day; the record month being March, 1911, 653 ft. being driven."

The Alpine tunnels referred to were driven by the European system, and this Laramie Tunnel was driven by the American system, but the American system as used at Laramie involves a distinct departure, in that, instead of columns being used for mounting the drills, a horizontal shaft bar was used similar to that carried by the Alpine tunnel carriage, but with the distinct difference that the Laramie bar was a light, simple apparatus, only 3 in. in diameter, mounting three hammer drills, as distinguished from percussive drills.

The main purpose in a carriage is to provide a means by which heavy drilling machinery may be used so as to do rapid work, and the main advantage of a carriage over the column system is that it enables the contractor to put his heavy machinery in the face and at work more rapidly than with the use of columns. If, however, heavy machinery is not required for the heading; that is, if the work

Mr. Saunders. can be done equally well by the use of light drills, then the necessity for a carriage is removed.

Now, it is a well-known fact that drills of light weight, which always means drills of smaller diameter of cylinder, will not accomplish as much work in heading driving as drills of heavier weight and larger diameter of cylinder. This is true as long as percussive drills are compared; that is, piston drills, machines where the cutting tool forms an extension of the piston rod and where this cutting tool and piston are reciprocated by the power. At Laramie, however, a different type of drill was used, machines of the hammer type, where the cutting tool is not attached to the piston rod but is held stationary in the hole and the piston is used to hammer the end of the steel. This type of drill is lighter in weight, and, owing to the absence of the percussive principle, there is very much less vibration or backkicking to its mounting, hence it is not necessary to provide heavy mountings and tunnel carriages, but the drills attached to a horizontal bar are simply lifted over the top of the muck by a gang of men, jacked in place, and the drilling begins at the top of the heading while the mucking is going on below.

In connection with this system, it must be borne in mind that water and air, injected through the steel into the hole, form a very important part of the equipment. Without it, it is doubtful if the holes could have been drilled with sufficient rapidity, and it has the added advantage of quieting the dust and improving the atmosphere.

In both the Loetschberg and Laramie Tunnels the headings were about 10 ft. wide, at least this was the width of the first Loetschberg heading. At Laramie the height was 8 ft. and at Loetschberg 6 ft. 6 in. This additional height at Laramie called for the drilling of more holes and the removal of about 25% more material. Although this is a handicap in some ways, it is partly compensated for by the greater space it gives in which to work.

On comparing these tunnels, it is striking to notice that at Laramie the actual drilling time was more than double that at Loetschberg. At Loetschberg the holes were drilled to a depth of only 4 ft., this being repeated for each 8-hour shift, while at Laramie the holes were much deeper, and it is readily understood that deep holes cannot be drilled at the average speed of shallow ones.

In the Loetschberg Tunnel there were drilled $13 \times 2 = 26$ holes $\times$ 4 ft., = 104 ft., and the actual drilling time was 2 hours, or 52 ft. of holes were drilled per hour. In the Laramie Tunnel there were 23×7.5 ft. = 172 ft., but the drilling time was 5 hours, so that the number of feet drilled per hour was $172 \div 5 = 34.5$ ft., as compared with 52 ft. If it were not that the other operations of the cycle were done with remarkable celerity, the time record for the Laramie Tunnel would not have been nearly as good as it was, and it is evident that if more time could have been saved, it would have been in the actual drilling. Mr. Saunders.

Lazarus White, Assoc. M. Am. Soc. C. E.—The Laramie-Poudre is the latest drainage tunnel in the United States, and was driven with record-breaking speed. These tunnels, in general, have small cross-sections, and the headings are of a convenient size. The Laramie Tunnel was driven by the contractors who constructed the Roosevelt Tunnel, and the progress there was bettered. At the east end an average monthly progress of 474 ft. was made for 19 months, or 525 ft. per month during the last year of work, with a maximum of 653 ft. for March, 1911, which for hard rock is far in excess of the progress on any other American tunnel. The heading of the east tunnel was driven up a 2% grade. At the west end, though the driving was down grade, the progress was much less, averaging about 300 ft. per month. Mr. White.

This tunnel has a cross-section of 7½ by 9½ ft., which is favorable for driving at the maximum speed. The methods are peculiar to the West, inasmuch as Leyner drills were used, and the firing was done with fuses. The wages were very high, and liberal bonuses were paid in addition. A great many holes were driven in the face, and they were heavily loaded with dynamite. The quantity of dynamite used is not given, but; judging from the data, it can be estimated at about 12 lb. per yd., though it would be interesting to have the author state it.

By comparison with tunnels driven in the East, for water-works and railroads, the conditions were extremely favorable. Eastern tunnels are much larger, necessitating the use of heading and bench, and as they are usually lined with masonry, they require more careful driving. The tunnels of the Catskill Aqueduct are circular or horseshoe shaped, and are lined with concrete from 12 to 18 in. thick. To keep down the quantity of excess concrete, it is necessary to drive carefully. The yardage to be handled per foot of tunnel is about four times that of the Laramie-Poudre Tunnel, so that getting rid of the muck is a serious problem when an attempt is made at rapid driving. The best record made on the Catskill Aqueduct was 530 ft. on the Wallkill Tunnel, which is described by Mr. Hulsart, and at various other tunnels, the progress has been from 300 to more than 400 ft. of complete tunnel per month.

To get the closest parallel to the Laramie-Poudre Tunnel, one has to compare it with the bottom headings as driven in Switzerland. It seems to the speaker that the methods are quite similar. In both cases the drills were mounted on horizontal bars, a great many holes were driven in the face, and these were heavily loaded with high

Mr. White. explosive. To save time in firing, the entire heading was shot by attaching fuses of different lengths, so as to give a succession of rounds. Iron plates were used from which to muck, and the men were paid a heavy bonus. In both cases very large ventilating plants were used, allowing the men to return to the face soon after firing. The parallel can be carried still further by comparing the use of hydraulic plants to furnish compressed air and electricity, near-by mountain streams being harnessed for this purpose.

In common with other Western tunnels, one feature which distinguished the Laramie-Poudre was the use of the Leyner drill. This drill is a purely Western product, invented and manufactured in Colorado. It is, in effect, a large hammer drill of hollow steel through which a combined stream of water and air is blown. This washes the hole clean and allows the steel to hit the solid rock directly with an uncushioned blow. This feature is said to account for the high drilling speed made in hard rocks. The Leyner is pre-eminently a heading drill, being much better at driving horizontal or up-holes than vertical or down-holes. It has the great virtue of drilling holes without making dust, and for this reason its use ought to be encouraged. It had already established a reputation at the time of starting the Catskill Aqueduct, and was tried out by three contractors. In all cases it was found to drill rapidly, to be very handy, and to eliminate dust; and, in common with hammer drills, to use less power per foot of hole than ordinary piston drills. The merits of the drill, however, were outweighed by its drawbacks, the principal one being that it appeared to be very delicate and unable to stand up, frequently breaking parts. A sufficient number of drills was not at hand or could not be kept in repair to do the work, so that, in all except one contract, they were superseded by ordinary piston drills. This contractor made considerable improvements, so that ultimately he claimed that the drills were working very well. The rights of manufacturing the Leyner drill have been bought by an Eastern firm, and the statement has been made that one is about to be put on the market which it is hoped will overcome the weaknesses of the former drill.

Firing with fuses, as practiced on the Laramie-Poudre Tunnel, effects a saving of a great deal of time, inasmuch as the shots are fired at one time. With the use of electric fuses it is generally necessary to fire at least three rounds, which consumes much time as the men must go back after each round and load or connect up the holes. This is the most disagreeable work connected with tunneling, because the headings are then necessarily very smoky. For some reason, the practice of firing with fuses in tunnels has been almost given up in the East, the general impression being that fuses are much more likely to miss fire. With fuse firing it is necessary to drill more holes and to use more powder. In case the cut does not pull to its depth, a great deal of time is likely to be lost. With electric firing it is common to reload the cuts until they are pulled to nearly their full depth. Ordinary fuses are stated to be much more reliable than they were at the time they were practically superseded in the East by electric fuses. Eastern tunnel men are not accustomed to their use, and Western contractors on the Catskill Aqueduct in most cases prefer the electric exploders. Mr. White.

Progress in tunneling in the United States is still far short of that made in Switzerland, but it would seem that it is as rapid as is consistent with economy. If the system in vogue in Alpine tunnels were followed in the United States, the much higher wages for labor would make the cost of tunneling almost prohibitive.

It would seem that if Americans can do all the work of installing a plant and establishing a camp in an almost inaccessible mountain region, and can drive a tunnel more than 2 miles long in less than 2 years, with a much smaller force than used abroad, and at a much smaller cost, they can be satisfied with the progress made in tunnel driving during the last few years.

O. J. Swensson, Jun. Am. Soc. C. E.—On one of the tunnels of the Catskill Aqueduct the method of drilling and shooting differed from that usually followed in the East. It was what is known as the stope method; that is, the bottom was drilled first, not as is usually done on a bench, but in a bottom heading about 100 ft. in advance of the top heading or stope. A timber platform was built under the stope, and extended about 50 ft. each way from the top face. This served the double purpose of a floor for the drilling and a place to catch the muck. This floor was of loose lagging, the pieces being removed to drop the muck into the cars. Thus the operations for the stope and bottom headings were carried on at the same time. Mr. Swensson.

In this method horizontal bars were used, not the vertical bars, because only a half section of the tunnel was to be driven. In the stope, where work was going on at the same time, ordinary tripod drills were used. Later, for the trimming and for the high points, the men used a long drill, with an extension rod but no tripod, termed a wall drill. In shooting the bottom heading, six cut holes in the center, with one riser, and about twelve to fifteen side rounds, were used.

At one time a method of using only one drill shift and two mucking shifts was tried, and resulted in very good progress. In trying to improve on this, a split shift was used. The drillers would go in and hole a round, while the muckers were digging out the muck back of the heading. After shooting, four or six muckers, termed "king muckers," as they were more experienced in that line of work and better workmen, were sent in and mucked down the headings close to

Mr. Swensson. the face, so that the drills could be set up. Then the men of the drill shift came back and were ready to drill again while the remainder of the heading was being cleared.

On one of the Catskill tunnels a very serious cave-in occurred about two years ago. The tunnel had been driven through the rock for about 1 200 ft. from the portal. The rock was hard, but soft mud seams were frequent. There was a very bad mud seam 200 ft. back from the face which it was intended to timber in, but, as the timber had been delayed in transportation, the seam was only braced by stulls. One night, about twelve o'clock, large pieces of rock began to drop from the roof and sides. The men were then warned to stop drilling and come away from the heading and had hardly gotten out when the roof caved in to a height of about 40 ft. above the invert. The proposed height of the tunnel is only about 17 ft. When the mucking shifts had removed all the caved-in material, heavy timbering was placed where the cave-in occurred, and the drillers were allowed to go back to work at the face. The space above the timbering was dry packed, and then piled up to the roof with cord-wood, provision being made for a great many grout pipes.

Evidently the timbering was not strong enough, for, shortly after all the dry packing and cord-wood had been placed it came down, with a great quantity of rock on top. The height of the second cave-in could not be determined, as there was so much muck. An attempt was made to tunnel through this second cave-in, a small drift being made through its center, in order to get into the heading and see that everything was all right there, and also to ascertain the length of the cave-in. It was deemed advisable to stop all drilling and shooting in the bottom heading.

The next step was to tunnel through this second cave-in, and to the proper cross-section. It was determined not to take out all the muck, as the quantity was not known, or what damage might be caused if it fell, but to tunnel through the heap by driving two small drifts, one on either side, up near the proper roof line. Then five segment arch rings of 12 by 12-in. timbers were put in on wall-plates. Only short wall-plates could be used, as great care had to be taken not to tear down the muck piles. Provision was made for a large number of grout pipes. After these timbers had been placed and wedged as tightly as possible, the mucking was carefully done and plumb posts were put in. This was done all the way through the cave-in, from 30 to 40 ft., without accident, although the work was very dangerous, some of the pieces of muck being very large.

Mr. Wait. B. H. Wait, Assoc. M. Am. Soc. C. E.—This paper is interesting, on account of the speed attained in tunneling and the conditions which made this possible. One thing which impressed the speaker was the distance of this tunnel from railroads or accessible points, and other features were the good plant and organization which the contractors and owners took the trouble to secure. Mr. Wait.

The methods used did not differ to any great extent from the ordinary ones in working headings of that size. In the exceptional speed attained, more was due to the good plant and organization, and the good rock encountered, than to the methods. A great many headings of this size have been driven by practically similar methods, but in such cases the organizations did not have the efficiency to make such speed.

The high quality of the explosive used increased the progress materially. The speaker believes that this high explosive could be used advantageously for a great many rocks.

Another material aid to progress was the efficient ventilating plant. Many contractors near New York City, where good plants could be easily obtained, do not seem to realize the fact that ventilation in tunnels is important, and that progress depends on it to a great extent. If a tunnel is not properly ventilated, there is an unnecessary loss of time after shooting, and a decreased efficiency in the workmen practically throughout the 24 hours.

A condition which worked to the advantage of the contractor was the limited extent of bad ground encountered, and the small amount of time lost through break-downs. The fact that only two days were lost during the two years' driving on this tunnel was remarkable, and speaks well for the contractor's plant and organization.

C. Raymond Hulsart, Assoc. M. Am. Soc. C. E.—The maximum length of tunnel driven in one month on the new Catskill Aqueduct for New York City was 523 ft., in September, 1910. This was on the Wallkill Tunnel, a siphon or pressure tunnel, 4.4 miles long, in Hudson River shale. The foregoing record was made in the north heading of Shaft 3, one of six shafts giving access to the tunnel. Mr. Hulsart.

The tunnel cross-section is circular with a required average diameter of 17 ft. As actually excavated, the average diameter is nearly 18 ft. The circular section was found to be a difficult one to excavate in the lower half without trimming. The upper half was excavated closely to line by the top heading, but the bench left tight sides in the lower half which were trimmed after the excavation for the heading and bench was completed. The cross-sectional area excavated by the heading and bench was 230 sq. ft., or 8½ cu. yd. per lin. ft., as compared with the 2½ cu. yd. in the Laramie-Poudre Tunnel.

The Hudson River shale formation consists of layers of sandstone and shale in well-defined beds from a few inches to a foot or more in thickness. The strike is northeast across the line of the tunnel at an angle of 60°, and the dip is southeast about 25 degrees. This rock

Mr. Hulsart. was fairly easy drilling, a 3¼-in. Ingersoll drill cutting about 10 ft. per hour, including changes of steel. There was little trouble in shooting the ground, the cut usually pulling with one shot. The explosive was 60% Forcite, the average quantity used being 3½ lb. per cu. yd. of heading and bench.

Shaft 3, the permanent drainage or unwatering shaft of the siphon, is about 340 ft. deep to the tunnel grade. It is offset 75 ft. from the tunnel and connected with it by a cross-drift. The hoisting equipment consisted of two balanced cages operated by a 150-h.p. Lambert electric hoist. The muck was moved from the heading in 1½-cu. yd. Koppel side-dumping tunnel cars, which were run on the cages and out to the dump. The average haul from the heading to the shaft for the month when the record was made was 2 300 ft. The loaded cars were braked down a 2% grade from the heading and hauled back by mules. A single track, of 30-in. gauge, was used, with the necessary turn-outs and an extra spur at the bench.

The general method of excavation was that of top heading and short bench. The quantities removed were about 5 cu. yd. per lin. ft. of heading and about 3½ cu. yd. per lin. ft. of bench. The drilling equipment consisted of four 3¼-in. Ingersoll drills, two mounted on columns and arms in the heading and two mounted on tripods on the bench. Compressed air was supplied from a central plant about a mile away; the air was compressed to 100 lb. at the plant and was supplied at about 85 lb. at the heading.

The tunnel force consisted of two drilling and three mucking shifts of 8 hours each. Each drilling shift drilled and shot a complete round of heading and bench, making two advances a day, which averaged 8.7 ft. or 17.4 ft. per day. The four hours between drilling shifts were utilized in scaling down and mucking back the heading and in setting up columns and drills. Two extra drillers and their helpers, assisted by a part of the mucking gang, did this work. They were usually able to have all the drills set up and one hole drilled when the regular drilling shift came to work.

The heading round consisted of 26 holes, as follows: six cut holes, 12 ft. deep, three on each side about 7 ft. between collars; six relief holes, 10 ft. deep, three on each side of the cut with two breaking-down holes above the cut; and an outside round of twelve 10-ft. holes. The heading was shot in three blasts: first, the cut of six holes; second, the relief and breaking-down holes; and third, the outside round of twelve holes. All the holes of each blast were shot simultaneously, with Victor electric fuses, from the 220-volt lighting circuit.

Each mucking shift consisted of one foreman and twenty-five muckers. This force put up runways, mucked the heading, wheeled the muck in barrows to cars at the face of the bench muck pile, about 100 ft., and shoveled the bench muck directly into the cars. Nearly

150 cu. yd. of solid rock, or 250 cu. yd. of muck, were handled per day. This made 2 cu. yd. of solid rock or 3.3 cu. yd. of muck per man (wheelers and shovelers). Mr. Hulsart.

This tunnel was driven for the Board of Water Supply of New York City by the Degnon Contracting Company. The large amount of muck handled and drilling done during this work was made possible only by the very fine organization and equipment of the contractor.

Burgis G. Coy, Assoc. M. Am. Soc. C. E. (by letter).—The discussions on the Laramie-Poudre Tunnel have brought out many points of interest to the writer, especially when comparing the methods of construction and rates of progress with those of tunnels in other parts of the world. Mr. Coy.

It seems to be the general conclusion that the shorter rounds give the best results and more economical work. This fact was also proven in the Laramie-Poudre Tunnel; the 7-ft. rounds proved more economical and gave greater progress than the 10-ft. rounds, unless the ground was exceptionally favorable, in spite of the fact that the number of set-ups was increased by about 40 per cent. The 7-ft. rounds also made smoother tunnel walls than the 10-ft. rounds, which would mean a saving of concrete if the tunnel were to be lined, and a lower coefficient of friction if left unlined. The direction of the tunnel was nearly parallel with the formation of the rock, which undoubtedly made it more difficult, both to drill and to shoot, than if it had been cross-cutting the formation.

Except where the ground was very soft, the round usually consisted of 21 holes, while in some cases, in extremely hard rock, this number was increased. Experiments proved that fewer holes did not break as well, often requiring a second shooting, and as the 18 holes drilled from the upper set-up could be drilled by the time the muck was out, very little if any time would be saved by drilling fewer holes. It was also found that the greater the number of holes, the smaller the muck was broken, and therefore the more easily and quickly removed. Very few holes failed to break bottom with one firing.

The writer's experience has been mostly with Leyner drills; therefore a comparison of the merits of Leyner drills with those of other makes will not be attempted, but some of the favorable points of this drill are its light weight, making it easily handled, and quick to set up even in close places, the absence of dust at the tunnel face even when drilling "up" holes, and the ease with which the drills can be changed. The holes are easily kept straight, even in broken and seamy ground, and there was almost no difficulty with stuck drills, even when using the 10- and 12-ft. steel.

Three men each made a monthly average of more than 60 lin. ft. of hole per 8-hour shift, the best averaging 61.86 ft.

Mr. Coy.

The time required for the various operations was as follows:

Exhausting smoke from face. .10	to	12 min.
Picking down roof and sides. . 5	to	10 min.
Jacking cross-bar in place.... 6	to	8 min.
Attaching drills, making hose and water connections.... 5	to	15 min.
Drilling from top set-up...... 3 hr.	to 4 hr.	15 min.
Dropping horizontal bar to lower position...........15	to	20 min.
Drilling on lower set-up...... 1 hr.	to 1 hr.	15 min.
Removing drills, cross-bar, hose, etc................15	to	20 min.
Blowing out holes, loading, and firing 20	to	25 min.
Ignition to explosion of last hole 8	to	8 min.

Total time required to complete cycle of operations is .... 5 hr. 24 min. to 7 hr. 28 min.

The following cost per foot is furnished by Mr. McIlwee, the contractor:

LABOR:	
Superintendent and foremen....................	$1.50
Drilling	4.47
Mucking, loading..............................	4.92
Tramming and dumping..........................	4.63
Track and pipe................................	0.47
Power-house	0.35
Blacksmithing	0.84
Repairs	0.47
Bonus to workmen..............................	1.75
SUPPLIES, ETC.:	
Maintenance of buildings, camps and fuel........	0.62
Machinery repairs.............................	0.12
Air drills and parts..........................	1.33
Picks, shovels, and steel.....................	0.84
Explosions	4.50
Lamps and candles.............................	0.42
Oil and waste.................................	0.38
Blacksmith's supplies.........................	0.53
Liability insurance...........................	0.81
Office supplies, telephone, and bookkeeping.......	0.86
Total..............................	$29.81

As the contractor was to receive a bonus of $300 per day for completing the tunnel before a specified date, some economy was sacrificed to speed of driving, and there is no doubt that the cost could have been lowered somewhat if the bonus proposition had not been considered. The contractor's bid was $32.50 per ft. and the bonus amounted to $63 000. Mr. Coy.

The total overhead charges, including power-plant, camp-buildings and furnishings, pipes, rails, etc., furnished by the company was approximately $120 000. Owing to the location of the work, the value of the plant was very small after the completion of the tunnel, as freight on pipes, rails, building material, etc., from the site to any market would cost more than their value. The entire plant, including all buildings and machinery at both ends, was sold for $10 000, leaving $110 000, or $9.73 per lin. ft., as the net overhead cost of the tunnel. The company purchased the plant and expects to use it to generate electricity to be transmitted to Greeley, Colo., and the machinery, other than that needed for electrical power development, is being taken away.

In all, 985 ft. of tunnel were timbered at a cost of $2.32 for material and $4.73 for labor. This work was done by the contractor at cost, plus 10 per cent.

During March, April, and May, 1911, 14 904 cars of muck, aggregating 9 107 cu. yd., were taken out. The total tunnel excavation, as called for in the contract, is 4 296 cu. yd., or a ratio of 2.11 cu. yd. measured in the cars to 1 cu. yd. paid for in place. This quantity includes all over break, which was not measured, therefore it does not represent the true swell in breaking up the rock.

In conclusion, the writer wishes to acknowledge his indebtedness to Mr. J. A. McIlwee for the information and figures he has furnished for the preparation of this discussion.

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TRANSACTIONS

Paper No. 1232

THE APPRAISAL OF PUBLIC SERVICE PROPERTIES AS A BASIS FOR THE REGULATION OF RATES.*

By C. E. Grunsky, M. Am. Soc. C. E.

With Discussion by Messrs. William Brokaw Bamford, James V. Oxtoby, Charles H. Higgins, Henry Floy, J. Martin Schreiber, William J. Boucher, R. D. Coombs, A. H. Van Cleve, W. Kiersted, and C. E. Grunsky.

Introduction.

It is the purpose of this paper to make clear the fact that appraisals of public service properties for rate-fixing purposes can be made, with advantage to all parties concerned, without deducting anything from the properly invested capital for depreciation. Incidentally, it will be pointed out that depreciation must, and how it should, be taken into account in estimating net earnings; that appreciation should be regarded as a reinvestment of earnings; and that there is and can be no definite basis for such elements of value as "going concern" and the like, unless operations are conducted under a restrictive franchise, that is, unless the franchise frees the public service corporation from outside control of its rates. No apology need be made for the elementary treatment of parts of the subject.

The appraisal of public service property for various purposes, such as taxation, regulation of rates, purchase, and limitation of indebtedness is receiving attention by many engineers and financial experts at the present time.

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* Presented at the meeting of June 5th, 1912.

Ideas in the matter of valuation, both in relation to what should be included in the valuation and to the valuation itself, are so diverse that a few words about the fundamental principles which should control when the appraisal of a public service property is to serve as a basis for fixing the rates to be charged for the service, are now offered in the hope that what is presented may aid in unifying methods of computing the desired earnings and in overcoming some of the difficulties into which the Courts are leading the experts.

The definite programme herein indicated for dealing with depreciation and amortization is the outcome of the writer's professional employment in California on appraisals, representing, at various times, the rate-payers, and, at other times, the owners of public service properties.

No attempt will be made to apply the conclusions reached to valuation for other purposes than the regulation of rates, nor to valuations in those cases in which special privileges have been granted, in which, therefore, the franchises are in the nature of an agreement or contract. Neither is it proposed to deal with any cases out of the ordinary, such as those in which the owner of a property was aided by bond issues, land grants, and the like. The main purpose will be kept in view of presenting a clear analysis of the fundamental problem, so that an understanding may be had of the principles which should prevail whenever an appraisal is made for rate-regulation purposes.

The Law in California.

The following is from the Constitution of the State of California (in effect January 1st, 1880):

"The use of all water now appropriated, or that may hereafter be appropriated, for sale, rental or distribution, is hereby declared to be a public use and subject to the regulation and control of the State, in the manner to be prescribed by law; provided, that the rates or compensation to be collected by any person, company or corporation in this State for the use of water supplied to any city and county, or city or town, or the inhabitants thereof, shall be fixed annually by the Board of Supervisors, or city and county, or city or Town Council, or other governing body of such city and county, or city or town, by ordinance or otherwise, in the manner that other ordinances or legislative acts or resolutions are passed by such body, and shall continue in force for one year and no longer. Such ordinances or resolutions shall be passed in the month of February of each year, and take effect on the first day of July thereafter.

"The right to collect rates or compensation for the use of water supplied to any county, city and county, or town, or the inhabitants thereof, is a franchise, and cannot be exercised except by authority of and in the manner prescribed by law.

"In any city where there are no public works owned and controlled by the municipality, for supplying the same with water or artificial light, any individual, or any company duly incorporated for such purpose under and by authority of the laws of this State, shall, under the direction of the Superintendent of Streets, or other officer in control thereof, and under such general regulations as the municipality may prescribe for damages and indemnity for damages, have the privilege of using the public streets and thoroughfares thereof, and of laying down pipes and conduits therein, and connection therewith, so far as may be necessary for introducing into and supplying such city and its inhabitants either with gas light, or other illuminating light, or with fresh water for domestic and all other purposes, upon the condition that the municipal government shall have the right to regulate the charge thereof."

The charter of the City and County of San Francisco (in effect January 8th, 1900) mentions, among the powers of the Supervisors:

"To fix and determine by ordinance in the month of February of each year, to take effect on the first day of July thereafter, the rates of compensation to be collected by any person, company or corporation in the City and County, for the use of water, heat, light or power, supplied to the City and County or to the inhabitants thereof, and to prescribe the quality of the service."

General Remarks.

The value of a revenue-producing property is determined ordinarily by its earning capacity; but this earning capacity, when it is to be used as a basis for valuation, must itself be determined with proper consideration of all attendant circumstances. It is not enough to compare receipts with current expenditures when estimating net returns. Account must be taken of the useful life of the property, because, in order that the property may be kept at a standard serviceability, parts, or all of it, must be replaced from time to time, unless indeed it happens to be of a character similar to real estate, having for all practical purposes a perpetual life. Such circumstances as appreciation in value which may arise from an advance of real estate values, or from other causes, must also be taken into account.

Knowing the earning capacity and the ordinary interest return expected from an investment, it becomes an easy problem in mathematics to capitalize earnings, it being understood that no necessary expenditure, present or prospective, has been overlooked, and that, in estimating useful life and the requirements of an amortization or a replacement fund, every factor, such as ordinary deterioration, inadequacy, and obsolescence due to advance in the arts, shall have been given due weight and consideration.

In the case of properties, however, the earnings of which are subject to regulation, an element of uncertainty concerning the amount of earnings may be introduced, rendering it impossible to use earnings as a basis for computing value.

Cases do occur in which franchises are exclusive, and in which rates for services rendered remain operative for long periods of time, in which, therefore, even under a system of rate regulation, it is possible to estimate earnings for long future periods and in which the earnings are determinable with sufficient precision to be used, in some measure at least, in determining value; but this is not so in California. There the system of annual rate fixing prevails, and the maximum rates allowed are presumed to be fixed so that under them there may be a fair return to the corporations on the value of the properties actually in use in rendering the service. Perhaps the use of the term, "value," in this connection is unfortunate, because it is not clear why "value," as ordinarily defined (which is not always synonymous with capital reasonably and properly invested), should be made the criterion of allowable earnings. It is reasonable to assume that the term, "value," in connection with the fixing of rates, has been used without prejudice to the rights of the owner of a public service property, and, therefore, some note will be taken, in what is here presented, of value as it may appear from different standpoints.

It is not necessary to state that, in a critical analysis of earnings, which go in part to an amortization fund and are in part distributable as a return on the investment, the rate of interest taken into account should be the same throughout. When it shall have been determined in any particular case what the earnings must be to yield the same rate of return as could be obtained from ordinary safe investments, then any desired addition as compensation for having undertaken the operation of the public service property, or for unusual risk, may be added.

USEFUL LIFE, VALUE, DEPRECIATION, AND THE AMORTIZATION FUND.

The general presentation of the problem will be simplified by assuming that the actual useful life or service of a plant, or part of a plant, conforms in every case exactly to its expectancy. That this is not in reality the case, and how it affects the conclusions which otherwise would be reached, will be referred to later.

In the presentation of the subject, this assumption has been strictly adhered to, and the rate of interest uniformly applied is 4% per annum.

Take the case of a plant, every part of which has a life of 20 years, all constructed at one time and owned by a prudent owner who sets apart, as an amortization fund, each year \$0.03358 for every dollar invested therein at an interest of 4% per annum. If the plant is one which will actually net 4%, then the apparent excess of the earnings, including amortization, over expenses, should be 4 + 3.36 = 7.36%, and the owner, in estimating the price at which he can sell without loss at the end of any period, as, for example, at the end of 10 years, would figure as follows (for each \$100 of original investment):

Investment	\$100.00
In the amortization fund: being the amount of a 10-year annuity of 3.358% at 4% interest..	40.30
Remaining value...........................	\$59.70

A prospective purchaser would figure that the plant should be worth at least as much as the annuity of 3.358% would amount to in the remaining years of the plant's life.

The amount of a 10-year annuity of \$3.358 is \$40.30, and to this amount he would add a sum determined from the excess earnings of the plant over a net earning of 4% during its remaining life on the part of the value which he has estimated, as explained, on the amortization or retirement fund basis.

The earnings being based on 4% of the originally invested capital, the increment of value covered by the amortization fund being \$40.30, on which 4% is \$1.609, for 10 years there will be earned an additional \$2.391 per annum, of which the present value is \$19.40. The purchaser, therefore, will conclude that he can invest with assurance of a 4% net return, \$40.30 + \$19.40 = \$59.70.

Of course, the same conclusion could have been reached by determining the present value of the earnings treated as an annuity of \$7.358 for the remaining 10 years, which, at 4% per annum, is \$59.70.

At the end of 10 years, the original owner, keeping for his own use the money in the replacement fund, will be satisfied to sell at \$59.70. The purchaser, content in this case with the assumed rate of interest of 4%, will be willing to pay this \$59.70, because, at the end of the plant's useful life, he will have recovered his investment with 4% interest compounded annually. He will then be under the necessity of replacing the plant, making a new investment of \$100, as the original owner would have been if he had remained in possession.

During the entire 20 years of usefulness the plant has been rendering adequate service. The sufficiency of the service is independent of, and bears no relation to, the useful life of the plant, nor to the fact that it was gradually deteriorating. During all periods of the plant's life, the depreciation of its physical elements was offset by the accumulation in the amortization fund.

Of course, it cannot be known just how, nor at what rate, the actual deterioration of a plant takes place. This may be rapid at some period of its life, and slow at another, but, as it is supposed, at all times during its life, to be adequately performing the service expected of it, this rate of decay is entirely immaterial.

In other words, the amortization may be determined without regard to the physical condition of a plant at any period of its life, provided, of course, that the plant fulfills the requirements of adequate service at all times of its life. For this reason it has become convenient to consider the actual, or the theoretical, accumulation in an amortization fund as the measure of plant depreciation with a consequent interchange of terms, which has led to a quite general use of the term, "depreciation," when designating the retirement of invested capital.

There is a clear distinction between amortization and replacement. The amortization deals with the repayment of the original investment. This may be in installments in uniform or unequal annual amounts, or in a lump sum at the end of useful life. The replacement may mean the substitution of a new identical plant, but at a cost dependent on new conditions, new prices of labor and material, or it may mean the substitution of new devices rendering equivalent service. In either event the replacement may be at a greater or less cost than the original cost, with, therefore, a corresponding increase or decrease of capital invested. The expenditure for replacement is amortization only to the extent that it retires capital already invested.

Perhaps, in referring to the worn-out part, the term "retirement" would be more applicable. In the sense in which "replacement" is used throughout this paper, except when otherwise explained, it is that part of the new investment in permanent construction which is equal to the capital theretofore invested in the parts which are discarded and replaced. Expenditures for new parts of a plant, which take the place of old parts which are retired for any cause, should be charged to replacement only to the extent of capital thus retired. Any excess of the expenditure for replacement over the cost of the discarded part of a plant should be treated as an addition to, and any less cost as a deduction from, the invested capital. The term, "replacement," used in the sense of retirement of invested capital, deals with the cost of the replaced part and not with that of the new equivalent installation. Theoretically, the amount which should go into an amortization fund should be estimated on the basis of invested capital or cost, and actual replacement should be made up out of this fund as far as the same may prove adequate.

Returning again to the case of the supposed valuation by a seller and by a purchaser of a plant with a 20-year useful life, at the end of a 10-year period; there is no need of assuming that an amortization fund has actually been created. In the case of both the original owner and a purchaser, the amortization annuity, instead of being actually placed in funds, may be otherwise invested.

When the owner of a plant which is yielding 4% per annum and nothing for amortization, sets apart, out of the 4%, an annual amount, also bearing interest at 4%, to meet its replacement at the end of the plant's useful life, he has invested not only the original cost of the plant, no part of which comes back to him in the annual 4% return, but also a gradually increasing sum which in the life of the plant will become adequate to replace it. At the end of the plant's usefulness, after replacing it with a new one, the total investment will be doubled without any increase of earning capacity, and the owner will have in effect lost his original investment.

It follows from this that a return of 4% per annum, without anything for amortization, on an investment in a perishable plant, when money is worth 4%, is inadequate. The excess of earnings over expenditures must be at least equal to the current interest rate on safe money investments plus an increment depending on the useful life of the plant. This increment must be such that, within the life of the plant, it will return to the owner his original investment.

Had the owner borrowed money for the construction of the plant, and were he paying interest on the borrowed money at 4%, this fact would be self-evident. The 4% earnings would then be required to meet interest payments, and, at the time when his plant has reached the end of its life and must be replaced with a new one, he would find himself, not only in debt for the original plant, but would have to duplicate the indebtedness for the replacement.

The amortization increment is ordinarily expected to appear in the earnings as that sum which, at compound interest during the life of the plant, will be adequate to retire the original investment.

To illustrate these points further, let it be supposed that ownership is represented by capital stock of a corporation. If the plant owned by the corporation earns just enough to net 4% without any allowance for amortization, the stock which at the outset was worth 100% will gradually decrease in value until, at the end of the plant's usefulness, it will be worth nothing.

The situation is quite different when the earnings net 4% plus an annual amortization increment. In this case, the stockholder receives 4% each year, and the amortization fund grows as the plant depreciates in value. The stock remains at par from the beginning to the end of the plant's usefulness, and the money in the fund at the end of the period is available either for distribution to the stockholders, being a return of the money advanced by them, or for reinvestment in a new plant to replace the original one.

Should a sale be made at any time while the plant is in service, the valuation of the plant would be made, as already explained, with due allowance for its depreciation, and, this value being recognized by a purchaser and the price paid, there would again be 100% available for distribution to the stockholders, the deficiency of the selling price being made up by the accumulation in the amortization fund.

In the case of inadequate earnings, the valuation of the plant by a purchaser would be at all times less than the value determined by deducting depreciation; in the case of adequate earnings the valuation would be, as already explained, capital invested (or the replacement cost) less depreciation. Yet, in either case, amortization being computed in the ordinary way for the full expectancy of the plant, the only fair valuation for rate-fixing purposes, in a spirit of fairness to both owner and rate-payer, would be a valuation at par without any deduction for depreciation.

Theoretically, then, a part of the earnings each year should be placed in an amortization fund as a repayment of capital invested, and this may be used for the replacement of the plant when it has reached the end of its life.

The accumulation of an amortization fund to be thus used, however, while theoretically sound policy, is a measure not always adopted in actual practice, particularly when the properties owned are of a complex character—when they are made up of numerous parts of various periods of probable usefulness. Municipalities, State, and National Governments, do not set apart funds for the replacement of worn-out or antiquated buildings, parts of water-works, street pavements, sewers, and the like, until occasion arises. They do not maintain funds at interest out of which to reconstruct their public works. The sinking fund required to retire bonds which may have been issued to construct these works originally must not be confounded with a replacement fund. The one may be necessary to pay for the works in the first instance, the other to maintain them for all time. The annual contribution to the sinking fund is a partial payment for the original work. The contribution to a replacement fund, in the case of a plant free from debt which is to serve without time limit, is for the purpose of perpetuating the work, because in that case the replacement fund, as far as it will go, or as far as it is required, will be used for making replacements.

Though it may be difficult to make satisfactory forecasts with reference to necessary reinvestments to replace discarded parts of a plant, the requirements for amortization, being based on cost, are usually readily determinable with some degree of precision.

Thus far, the plant considered is assumed to have been constructed and put into use all at once, and is of such a character that all its parts have the same life. The same principles will apply when a plant is made up of many elements or parts having various periods of usefulness.

It is again possible to determine for each part the amortization fund or the replacement fund annuity, and from the annuities thus determined to estimate what the minimum earnings should be to prevent loss.

The following problem presents itself: In the case of a plant of gradual development, but of full growth and mature age, the useful life of all the parts of which is n years, it is desired to know what amount is in the amortization fund at any time, that fund being assumed to receive such an increment at the end of each year that, during the life of the several parts of the plant, this annuity, with interest, will amount to the original cost of these parts.

Being composed of a large number of elements—each year having added new ones—the addition to it per year will be taken at one-nth of the total plant as it stands at the end of the nth year.

For each dollar invested in the first year, there will be \$1 invested in each succeeding year, and for each dollar thus invested there will be n dollars of total investment.

Let i represent the rate of interest per year, and a represent the annual contribution to the amortization fund for each dollar invested.

Assume this to be available at the end of each year.

Then na will be, after n years, the annual contribution to the amortization fund for each dollar invested.

Let m represent any number of years greater than n.

During the first n years, after beginning the construction of the plant, there will be no replacements, and the amortization fund continues to grow. At the end of the nth year the replacement requirement, assuming permanency in character and cost, will be \$1 for each dollar of annual investment, and this replacement requirement will continue at this rate thereafter.

At the end of the nth year the amortization fund will contain:

For each dollar invested the first year:

$$a\left(\frac{100+i}{100}\right)^{n-1}+a\left(\frac{100+i}{100}\right)^{n-2}+\ldots+a \text{ dollars}$$

Or,

$$\frac{100\,a}{i}\left[\left(\frac{100+i}{100}\right)^{n}-1\right] \text{ dollars}$$

For each dollar invested the second year:

$$a\left(\frac{100+i}{100}\right)^{n-2} + a\left(\frac{100+i}{100}\right)^{n-3} + \ldots . + a \text{ dollars}$$

Or,

$$\frac{100\,a}{i}\left[\left(\frac{100+i}{100}\right)^{n-1} - 1\right] \text{ dollars}$$

For each dollar invested the nth year: a dollars

Less the \$1 replacement requirement at the end of the nth year.

Therefore, the total amount in the sinking fund at the end of the nth year, after deducting the \$1 replacement requirement of that year:

$$S_n = \frac{100\,a}{i}\left[\left(\frac{100+i}{100}\right)^{n} - 1 + \left(\frac{100+i}{100}\right)^{n-1} - 1 + \ldots . + \frac{100+i}{100} - 1\right] - 1$$

$$S_n = \frac{100\,a}{i}\left\{\frac{100}{i}\left[\left(\frac{100+i}{100}\right)^{n+1} - \frac{100+i}{100}\right] - n\right\} - 1$$

$$S_n = \frac{10\,000\,a}{i^2}\left[\left(\frac{100+i}{100}\right)^{n+1} - \frac{100+i}{100} - \frac{n\,i}{100}\right] - 1$$

There will be in the sinking fund for each dollar invested:

At the end of the $(n+1)$st year:

$$S_{n+1} = S_n\left(\frac{100+i}{100}\right) + n\,a - 1.$$

At the end of $(n+2)$d year:

$$S_{n+2} = S_n\left(\frac{100+i}{100}\right)^2 + (n\,a - 1)\left(\frac{100+i}{100}\right) + n\,a - 1.$$

At the end of the $(n+3)$d year:

$$S_{n+3} = S_n\left(\frac{100+i}{100}\right)^3 + (n\,a - 1)\left(\frac{100+i}{100}\right)^2 + (n\,a - 1)\left(\frac{100+i}{100}\right) + n\,a - 1.$$

And so on, and at the end of the mth year:

$$S_m = S_n\left(\frac{100+i}{100}\right)^{m-n} + (n\,a - 1)\left(\frac{100+i}{100}\right)^{m-n-1} + (n\,a - 1)\left(\frac{100+i}{100}\right)^{m-n-2} + \ldots\ldots + n\,a - 1.$$

Substituting the value of S_n and summarizing the series:

$$S_m = \frac{10\,000\,a}{i^2}\left[\left(\frac{100+i}{100}\right)^{m+1} - \left(\frac{100+i}{100}\right)^{m-n+1} - \frac{n\,i}{100}\left(\frac{100+i}{100}\right)^{m-n}\right] - \left(\frac{100+i}{100}\right)^{m-n} + \frac{100}{i}\left[\frac{(100+i)^{m-n} - 100^{m-n}}{100^{m-n}}\right](n\,a-1).$$

In this form the formula is convenient for use, being applicable to any value of n and any rate of interest.

For the interest rate of 4% per annum, that is, for $i = 4$, the formula becomes:

$$\left.\begin{array}{l} S_m = \dfrac{a}{0.0016}\left[1.04^{m+1} - 1.04^{m-n+1} - 0.04\,n\,(1.04)^{m-n}\right] - 1.04^{m-n} \\ \qquad + \dfrac{1.04^{m-n} - 1}{0.04}(n\,a-1). \end{array}\right\}$$

For $m = n$, this formula becomes:

$$S_n = \frac{a}{0.0016}\left(1.04^{n+1} - 1.04 - 0.04\,n\right) - 1.$$

Applying the foregoing formulas to various periods of life, but adhering to an interest rate of 4%, the following amounts in amortization funds at various times are to be noted:

For a plant of full growth, all parts of which have a useful life of 5 years: $n = 5$, $a = 0.1846$, and the total invested capital is equal to five times the annual investment:

Years.	Amount in amortization fund for each $1 of annual investment.	Amount in amortization fund in percentage of total investment.
At the end of 5	$1.92	38.4
" " " " 10	1.93	38.6
" " " " 15	1.92	38.4
" " " " 20	1.92	38.4

For a plant of full growth, all parts of which have a useful life of 10 years: $n = 10$, $a = 0.08329$, and the total invested capital is equal to ten times the annual investment:

Years.	Amount in amortization fund for each $1 of annual investment.	Amount in amortization fund in percentage of total investment.
At the end of 10	$4.17	41.7
" " " " 15	4.18	41.8
" " " " 20	4.18	41.8
" " " " 30	....	....
" " " " 40	4.18	41.8
" " " " 80	...	....

For a plant, all parts of which have a useful life of 20 years: $n = 20$, $a = 0.3358$, and the total invested capital is equal to twenty times the annual investment:

Years.	Amount in amortization fund for each $1 of annual investment.	Amount in amortization fund in percentage of total investment.
At the end of 20	$8.21	41.1
" " " " 30	8.20	41.0
" " " " 40	8.19	41.0
" " " " 80	8.21	41.1

For a plant of full growth, all parts of which have a useful life of 40 years: $n = 40$, $a = 0.01052$, and the total invested capital is equal to forty times the annual investment:

Years.	Amount in amortization fund for each $1 of annual investment.	Amount in amortization fund in percentage of total investment.
At the end of 40	$14.47	36.1
" " " " 60	14.40	36.0
" " " " 80	14.43	36.1

In the foregoing mathematical analysis, a plant has been assumed which has already reached an age exceeding the useful life of its parts, and has reached its full growth.

The same formulas will apply in the case of any plant of mature age and gradual growth, even when the growth is still being extended, because in this special case the plant may be regarded as made up of two groups of parts, one embracing all those having an age of n years or less, and the other those parts which are more than n years of age, and for both these groups the formulas apply.

It is noteworthy, in the assumed case of a plant of full growth made up of numerous parts, that the amortization fund should bear a nearly uniform relation to capital invested, whatever the life of the plant may be. For such a plant constructed progressively, all parts of which have a useful life of 5 years, the amortization fund will, at any time after 5 years, contain an amount equal to about 38% of the cost; for a similar plant with a 20-year life, 41%; and for a similar plant with a 40-year life, 36 per cent.

The accumulation, therefore, in amortization funds, under certain hypothetical conditions, when plants are of progressive growth and mature age and are composed of numerous parts, at 4% interest, amounts to about 40% of the invested capital. In reality, however, there is never absolute agreement between the actual useful life of every part of the plant and its expectancy. The formulas are never strictly applicable. The requirements for replacement may begin long before the expectancy is attained, and if met from the amortization fund will check its growth.

The non-existence of a fund in the full amount indicated by mathematical and theoretical considerations, therefore, does not always show that it has been distributed as profit, nor yet that there has been an intentional waiver of the right to have the earnings cover a fair amortization allowance.

Furthermore, if the annual amortization increment is immediately applied in repayment of invested capital, the same no longer bears interest. Treated as an annuity, interest may be compounded only as long as the fund remains practically in escrow for its intended purpose, that is, for complete retirement at the end of the useful life of the plant. Interest ceases to accumulate the moment the fund is applied to retire the investment in whole or in part. Consequently, if the amortization be determined from amortization tables based on expectancy, and be covered by the earnings from year to year, even though the amortization increment as earned be reinvested in the property, it cannot rightfully be classed as a repayment of invested capital until the end of the period of expectancy. If so applied at any earlier date, a new amortization annuity, based on the remaining life, must be computed.

If, nevertheless, the amortization annuity as originally determined be deducted from the investment from year to year, the result will be incomplete amortization. In the case of a 40-year life, the amortization of the invested capital would be only $42.08 of each $100, and there would still remain $57.92 to be made good.

These facts make clear the point which is to be emphasized, that, whenever amortization is based on annuities bearing compound interest, the appraisal for rate-fixing purposes must be of the entire investment without reduction for depreciation.

The foregoing mathematical demonstration that the accumulation in an amortization fund for a plant of mature age should amount to a considerable sum, confirms a conclusion which can be reached in a more direct way.

In the assumed case of a plant which has a life of n years, and of which one-nth has been constructed each year, after n years there will have to be replaced one-nth thereof each year. Because the annual investment in the installation has been uniform, there will be, for each dollar invested per year, a total investment of n dollars.

The annual replacement after n years, for each dollar annually invested, will be \$1. If now the annuity to replace the several parts of the plant in n years is a for each dollar of the annual investment, then after n years, the annual amount received as annuity will be $a\,n$, and this will fall short of meeting the actual expenditure by an amount expressed by $(1 - a\,n)$ which, at 4% per annum, is the interest on $\frac{100\,(1 - a\,n)}{4}$ dollars; or, expressed in percentage of the cost, is $\frac{100^2\,(1 - a\,n)}{4\,n}$ per cent. of the total investment in the plant.

For a plant not subject to further growth, with a uniform useful life of all its parts, and constructed progressively, there will be needed, at 4% interest, to supplement the deficient amortization annuity:

When the useful life is 5 years:

$$\frac{100^2\,(1 - 0.923)}{20} = 38.5\% \text{ of the total replacement cost.}$$

When the useful life is 10 years:

$$\frac{100^2\,(1 - 0.8329)}{40} = 41.3\% \text{ of the total replacement cost.}$$

When the useful life is 20 years:

$$\frac{100^2\,(1 - 0.6716)}{80} = 42.3\% \text{ of the total replacement cost.}$$

When the useful life is 40 years:

$$\frac{100^2\,(1 - 0.4208)}{160} = 36.2\% \text{ of the total replacement cost.}$$

These figures are in substantial agreement with those resulting from the first analysis. They show that, in order to make an annual allowance, estimated by the annuity amortization fund method, adequate to keep a plant, of the kind assumed, in good condition, there must be allowed to accumulate and be kept always on hand a fund at

4% interest which, for expectancies of from 5 to 40 years, is somewhere near 40% of the replacement cost of the plant.

Some such amount, depending on the expectancy, represents the accumulation of the annuities during that period of the plant's life during which no replacements were necessary. If the annual allowance for maintenance has been in the past based on the requirements of operation and repair without surplus to meet future replacements, then the current allowance for amortization or replacement should not be determined by the amortization fund annuity method, but should be otherwise determined, as shown subsequently.

When, in other words, opportunity has not been given to accumulate the 40% (approximately), for ordinary periods of useful life of perishable properties, of the invested capital, then any amount estimated from amortization tables on the original full period of useful life will fall short of the real replacement requirement. To illustrate this point, let it be assumed that a conduit, such as a cast-iron pipe, used for any purpose, has a length of 40 miles. Let it be also assumed that the pipe is not being further extended, that the expectancy of this pipe is 40 years, and that it was constructed progressively, 1 mile each year. It took 40 years to install the pipe, and at the end of this time the first mile of pipe laid was ready for replacement—it had served its time. Each year thereafter, 1 mile of pipe has to be replaced, and the replacement at this rate will continue indefinitely. The annual replacement expenditure during the first 40 years is nothing, but, thereafter, it is the cost of installing 1 mile of pipe. If prices of labor and material have remained constant, and if conditions have otherwise remained as they were when the first mile of pipe was laid, then the annual replacement expenditure will be one-fortieth of the total amount invested in the pipe line.

Provision for this replacement must be made if the pipe is to continue in service. If, now, the extension of the pipe progresses beyond the 40-year period at the same rate, before assumed, of 1 mile per year, there will be no changes in the annual replacement requirement during a second period of 40 years, but at the end of this second period—at the end of 80 years—there will be 80 miles of pipe in service, and thereafter during the third 40-year period there will have to be replaced annually 2 miles of pipe, or one-fortieth of 80 miles, or twice the amount of pipe extension per annum.

It is possible, by such analysis, when a plant is of progressive growth and has attained an age exceeding the life of its perishable parts, to prescribe a rule for determining the replacement requirement; but it must be remembered that a rule thus determined can be strictly correct only for the hypothetical case of service in exact conformity with the assumed probable life, and that a rule thus determined may require some modification, as hereinafter explained.

For each group of parts having the same length of life, there is to be determined: first, the average annual capital invested, using, however, replacement cost instead of the actual investment; and second, the full number of times that the age of the plant is greater than the useful life of the particular group of parts under consideration. The replacement requirement (for the hypothetical case, in which actual service conforms throughout with the assumed probable life) is then ascertained by multiplication.

A pipe line may again serve as an illustration: Suppose it is desired to know the replacement requirement for a pipe line 300 miles long, which has been extended 2 miles each year, the age of the oldest portion of which, therefore, is 150 years.

The life of the pipe being taken at 40 years, the full number of times this is contained in 150 years is three. The annual replacement requirement will be three times two, or 6 miles of pipe.

The 6 miles of pipe requiring replacement were constructed 40 years ago, and the conditions under which this was done may have been materially at variance with those prevailing at the time of their replacement. Consequently, in the determination of the replacement requirement, expressed in dollars instead of in miles of pipe, the replacement cost of the system and not the original cost of capital invested should be taken into account. Expressed as a percentage of the total length of pipe in service, or of the total cost of replacing the entire pipe line, this would be 2 per cent.

By the annuity method of computation, in the selected illustration, the allowance for replacement would be 1.052% of the cost of the system, which is barely more than one-half of the actual requirement, and this allowance, as already explained, would only then be justified if amortization had covered the entire period in the life of each part of the pipe during which there was no expenditure for replacements, so that the inadequate annual allowance could be supplemented by the earnings of an accumulated amortization fund.

In a plant which is made up of a multiplicity of parts of various periods of usefulness, those which have the same expectancy should, as before stated, be grouped together. For each group, the replacement requirement can then be estimated separately, and from the several amounts thus ascertained the total requirement is determined.

The rule previously laid down for a hypothetical case is not strictly applicable under the conditions as they actually present themselves. There can be no absolute conformity between the assumed period of usefulness of any part of a plant and the time during which it actually proves useful.

The probable useful life or expectancy is merely the average life, which is often not reached and is just as often exceeded. Thus, again referring to the pipe line, it is to be assumed that while some of it may serve beyond the average period of usefulness of such pipe, other parts thereof, from one cause or another, will require replacement early in its life. Consequently, any rule such as that previously laid down, which indicates a uniform replacement requirement in successive periods, with a sudden rise in the requirement at the beginning of each new period, if the plant be one that is steadily growing, will require some modification.

The simplest modification of the foregoing rule is to assume gradual changes in the annual replacement requirement as the age of the plant increases, instead of the sudden changes, and then to call this requirement at all times inversely proportional to the useful life of any group of parts. This is sometimes referred to as the "straight-line" method. It might with equal propriety be called the direct percentage method, as the inverse ratio is usually expressed in percentage.

Under this direct percentage method, there would be allowed 2.5% per annum of the replacement cost of all parts of a plant having a 40-year life; 3.33% per annum of the replacement cost of all parts having a 30-year life; 5% per annum of the replacement cost of all parts having a 20-year life, and so on.

This method, applied to the hypothetical case of a pipe line, constructed and extended 1 mile per year, and each mile thereof having a useful life of exactly 40 years, would, at the end of the fortieth year, make the replacement requirement 2.5% per annum, or 1 mile of pipe. At the end of the sixtieth year, the requirement thus determined would be 2.5% of the 60 miles of pipe then in service, or 1.5 miles of pipe. This would be 50% in excess of the amount actually replaced, which at that time would be only 1 mile. This would also apply for any time before the pipe first laid has reached the limit of its usefulness, as at 20 years. In the assumed case there is no replacement requirement at 20 years; yet the straight percentage method indicates 2.5% of 20 miles of pipe, or 0.5 miles of pipe. It follows from this illustration that the straight-line, or direct percentage, method, applied to an estimated total cost of replacement, would give results somewhat too high.

By further analysis of this problem, the following formulas have resulted, which are free from this objection and fulfill every ordinary requirement. In devising these formulas, the fact was taken into account that there may be some replacement requirement in the early years of a plant's life, and that this requirement gradually increases. These formulas apply strictly only to plants which have been developed gradually and are being extended at a uniform annual rate.

Using the notation already introduced, and designating with R the total cost of replacing the group of items, the probable useful life of which, when new, was n years, and with $e =$ the average annual cost of extensions, the formulas are:

For m less than n: $r = \frac{m\,e}{2\,n}$, or $= \frac{R}{2\,n}$.

For m greater than n: $r = \frac{R}{n} - \frac{e}{2}$.

For very large values of m in relation to n (n being the years of probable usefulness), the value of this expression approaches $\frac{R}{n}$, which is the mathematical equivalent of the straight-line, or direct percentage, method.

However desirable it might otherwise appear to introduce a method of computing the replacement requirement by recourse to amortization tables, to do this equitably, in the case of a complex plant, is usually difficult, if past earnings have been inadequate to supply the proper amortization increment. In such cases the use of some formula, as above noted, for probable replacement requirement is to be recommended.

This method is strictly equitable from the standpoints of both the owner and the ratepayer. That this must be so will be seen on reflection.

The annuity or ordinary method of retirement may be regarded as an installment method. Under the replacement method an exactly equivalent lump sum, "the amount of the annuity," takes the place of the installments. If the installments are forthcoming as they are due, then the annuity method is adequate. If they are not paid, then recourse must be had to the lump sum or replacement method as above described.

It is perfectly reasonable, moreover, to assume, unless there is evidence to the contrary, that the method of estimating amortization requirements, which prevails in any case, has been introduced deliberately. The owner of the public service property may be perfectly willing to waive the annuity payments if he knows that what they will amount to, that is, the actual annual replacement, will be covered by the gross earnings when the time comes for discarding parts of his plant. In other words, he may be willing to accept the amount of an annuity in lieu of the annuity itself; and the rate-payer may desire such an arrangement, because, in the early days of the plant's life, he may not be able to pay a sufficient amount for the service to cover the amortization annuity. It must be remembered, however, that such an arrangement burdens the future rate-payer to some extent for the benefit of the rate-payer in the early days of a plant's life. This is the same idea as the one which prompts some engineers to add early losses to the valuation as a part of that intangible value which is usually called "going concern."

It follows directly from the foregoing that there may be cases in which, even though it be found proper to allow the full actual average annual replacement, the appraisal for rate-fixing purposes should still be the entire investment without any reduction for depreciation. This will be the case whenever it can be shown that past earnings were inadequate to provide an amortization fund.

The Expectancy.

Whether the plan of making the annual replacement allowance conform to the annual actual replacement requirement, as determined by formula, be followed, or whether either of the other two methods be adopted (the direct percentage method or the annuity method), due regard should be had, in fixing the expectancy, to the circumstances under which the plant is being operated and has been operated in the past.

Such disasters as the fire and earthquake of 1906, which suddenly put out of service large portions of the public service plants of San Francisco, which would otherwise have remained useful, may properly be taken into account, as noted hereafter, in estimating the probable useful life of any part of a plant.

It must not be expected, however, that the replacement increment, by whatever method determined, will in any year exactly meet the actual replacement requirement of that year. When some unusually costly part of the plant goes out of use and must be replaced, a single item of the replacement expense may greatly exceed the annual replacement allowance, while, on the other hand, whole series of years are to be expected in which the actual expenditure for replacement will fall below the allowance for replacement.

In the long run, if all assumptions have been properly made, there should be neither gain nor loss resulting from the allowance for replacement.

Before leaving this subject, it may be well to illustrate the fact that, when the age of a growing plant is many times greater than the useful life of a class of parts, the error made in applying the direct percentage method of computing replacement requirement will be small, and may ordinarily be disregarded.

In a plant which is 63 years old, for example, and has been extended at a uniform rate, those parts which have a useful life of 5 years should, according to the correct formula, be 19.2% per annum, while, on the assumption that all the parts having a 5-year life have been replaced, or have been put in new at a uniform rate during the preceding 5 years, the replacement requirement would be figured at 20 per cent.

Absolute accuracy cannot be hoped for, whatever the method of calculation, because the premises assumed as the basis for formulas are never exactly realized. Generally, however, under consideration of all circumstances, a reasonable approximation, either by the direct percentage method or by some formula similar to those previously laid down for a special case, can be made of the actual average annual replacement requirements, whenever the amortization annuity method of retirement does not prevail.

While the process of determining the annual requirement for replacement appears to be simple, it is, as previously intimated, made somewhat difficult and uncertain of application owing to the incomplete information available, from which to estimate the useful life of a plant or of the many parts which make up the whole. Many circumstances are to be taken into account in determining useful life, for this depends not only on deterioration by natural processes of decay, or wearing away by use, but also on inadequacy resulting from growing demand upon the plant; also on inadequacy or obsolescence resulting from changes in processes of manufacture, or from the use of new and better types of machines and appliances, and the like; and also destruction by unforeseen agencies, such as fires and earthquakes, landslides, floods, and the like. In these matters, past experience is the best guide, and, as already stated, should be given weight in some measure in assigning probable life to the parts of a plant.

In the case of gas-works, for example, the life of generators is shortened by the advance made in the art of gas manufacture. Within the last few decades, because of the high price of coal, the moderate price of oil, the local abundance of oil, and the introduction of new processes, the art of gas manufacture in California has been revolutionized. Old processes, are, for the time being, classed as obsolete, and generators and other parts of gas-works have gone out of use, in some cases, almost before their installation was completed.

Amortization and Annuity Tables.

The following tables have been prepared to illustrate certain principles, and no attempt has been made to give the figures therein presented that degree of accuracy which is usually looked for in amortization tables.

Table 7 is derived from the values noted in Tables 2, 4, and 6. It is the result of a multiplication of 100 times the figures in the column, "Remaining Value," with the figures in the next to the last column of each of these three tables.

The information contained in the tables is also presented in the curves in Figs. 1 and 2. Attention is directed to the fact, appearing in Table 7 and in Fig. 2, that the amortization increment required to retire the remaining value in the remaining life increases from year to year.

TABLE 1.—AMORTIZATION AND ANNUITIES. 5-YEAR EXPECTANCY.

Interest at 4 per cent. Annuities applied at the end of each year.

At the end of year.	ANNUAL AMORTIZATION INCREMENT FOR EACH DOLLAR INVESTED = $0.1846.			Annuity which will amount to $1 in the remaining life.	* Amount of an annuity of $0.20 in remaining life.
	Amount in amortization fund.	Value remaining in the physical properties.	Amount of the annuity in the remaining life.		
	$0.000	$1.000	$1.000	$0.1846	$1.083
1	0.185	0.815	0.785	0.2355	0.849
2	0.377	0.623	0.577	0.3204	0.624
3	0.576	0.424	0.376	0.4901	0.408
4	0.784	0.214	0.184	1.000	0.200
5	1.000	0.000	0.000	......	.....

* The annuity here noted is $1 divided by the expectancy.

APPRAISALS WITHOUT DEDUCTION FOR DEPRECIATION.

In determining the part of the investment on which the investor in public service properties should be allowed a reasonable income, all attendant circumstances must be duly considered. It may be stated, however, that, apart from the determination of the rate of interest which should result from the investment, it will be strictly equitable and fair to consider the public service corporation as the agent of the State or municipality, as the case may be, and to determine in what situation the State or municipality would have found itself had there been no intermediate owner or public service corporation.

Let it be assumed that the owner of a public service plant has made his investment under good expert advice, and that the plant is in every respect the same as, or equal to, what the people could have constructed for themselves. Let it be further assumed that the plant is free from debt, and that it and all its parts have a probable useful life of n years. The owner will then be entitled:

First.—To a reasonable interest on his investment;

Second.—To operating expenses;

Third.—To maintenance and repair expenditures;

Fourth.—To an annuity which, in n years, at the ordinary rate of interest, will amount to his investment.

TABLE 2.—AMORTIZATION AND ANNUITIES. 10-YEAR EXPECTANCY.

At the end of year.	ANNUAL AMORTIZATION INCREMENT FOR EACH DOLLAR INVESTED = $0.833.			Annuity which will amount to $1 in the remaining life.	* Amount of an annuity of $0.10 in remaining life.
	Amount in amortization fund.	Value remaining in the physical properties.	Amount of the annuity in the remaining life.		
	$0.000	$1.000	$1.000	$0.08329	$1.201
1...........	0.083	0.917	0.881	0.09449	1.058
2...........	0.170	0.830	0.766	0.10853	0.921
3...........	0.260	0.740	0.658	0.12661	0.790
4...........	0.354	0.646	0.552	0.15079	0.663
5...........	0.451	0.549	0.451	0.18463	0.542
6...........	0.552	0.448	0.354	0.23550	0.425
7...........	0.658	0.342	0.260	0.32036	0.312
8...........	0.767	0.233	0.170	0.49020	0.204
9...........	0.881	0.119	0.083	1.000	0.100
10...........	1.000	0.000	0.000	....	.....

* The annuity here noted is $1 divided by the expectancy.

TABLE 3.—AMORTIZATION AND ANNUITIES. 15-YEAR EXPECTANCY.

At the end of year.	ANNUAL AMORTIZATION INCREMENT FOR EACH DOLLAR INVESTED = $0.0499.			Annuity which will amount to $1 in the remaining life.	* Amount of an annuity of $0.066 $\frac{2}{3}$ in remaining life.
	Amount in amortization fund.	Value remaining in the physical properties.	Amount of the annuity in the remaining life.		
	$0.000	$1.000	$1.000	$0.04994	$1.335
1...........	0.050	0.950	0.913	0.05467	1.219
2...........	0.102	0.898	0.830	0.06014	1.008
3...........	0.156	0.844	0.750	0.06655	1.002
4...........	0.212	0.788	0.673	0.07415	0.899
5...........	0.270	0.730	0.600	0.08329	0.800
6...........	0.331	0.670	0.529	0.09449	0.706
7...........	0.394	0.606	0.460	0.10853	0.614
8...........	0.460	0.540	0.394	0.12661	0.527
9...........	0.529	0.471	0.331	0.15079	0.442
10...........	0.600	0.400	0.270	0.18463	0.361
11...........	0.673	0.327	0.212	0.23550	0.283
12	0.750	0.250	0.156	0.32036	0.208
13...........	0.830	0.170	0.102	0.49020	0.136
14...........	0.913	0.087	0.050	1.000	0.067
15...........	1.000	0.000	0.000	....	......

* The annuity here noted is $1 divided by the expectancy.

If it be now supposed that the owner actually received these amounts, estimated on a proper basis, and that he allows the annuity to accumulate so that amortization will be an accomplished fact at the end of n years, then, as he has command of the amortization fund, he will have a decreasing amount of capital actually tied up in the plant. This decreasing capital or remaining value of the plant is the complement of the growing amortization fund. This fund is supposed to be held

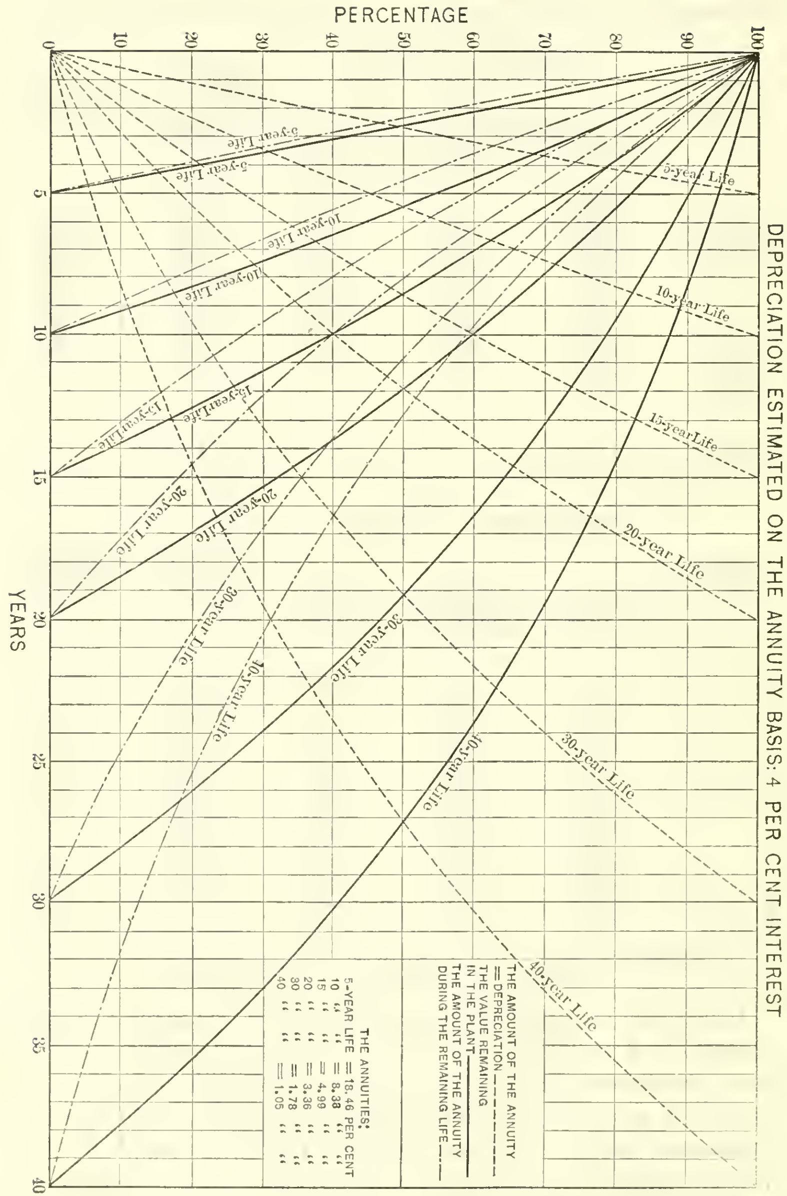
DEPRECIATION ESTIMATED ON THE ANNUITY BASIS: 4 PER CENT INTEREST
PERCENTAGE
0
10
20
30
40
50
60
70
80
90
100
YEARS
5
10
15
20
25
30
35
40
5-year Life
10-year Life
15-year Life
20-year Life
30-year Life
40-year Life
THE AMOUNT OF THE ANNUITY = DEPRECIATION
THE VALUE REMAINING IN THE PLANT
THE AMOUNT OF THE ANNUITY DURING THE REMAINING LIFE
THE ANNUITIES:
5-YEAR LIFE = 18.46 PER CENT
10 " " = 8.38 " "
15 " " = 4.99 " "
20 " " = 3.36 " "
30 " " = 1.78 " "
40 " " = 1.05 " "

FIG. 1.

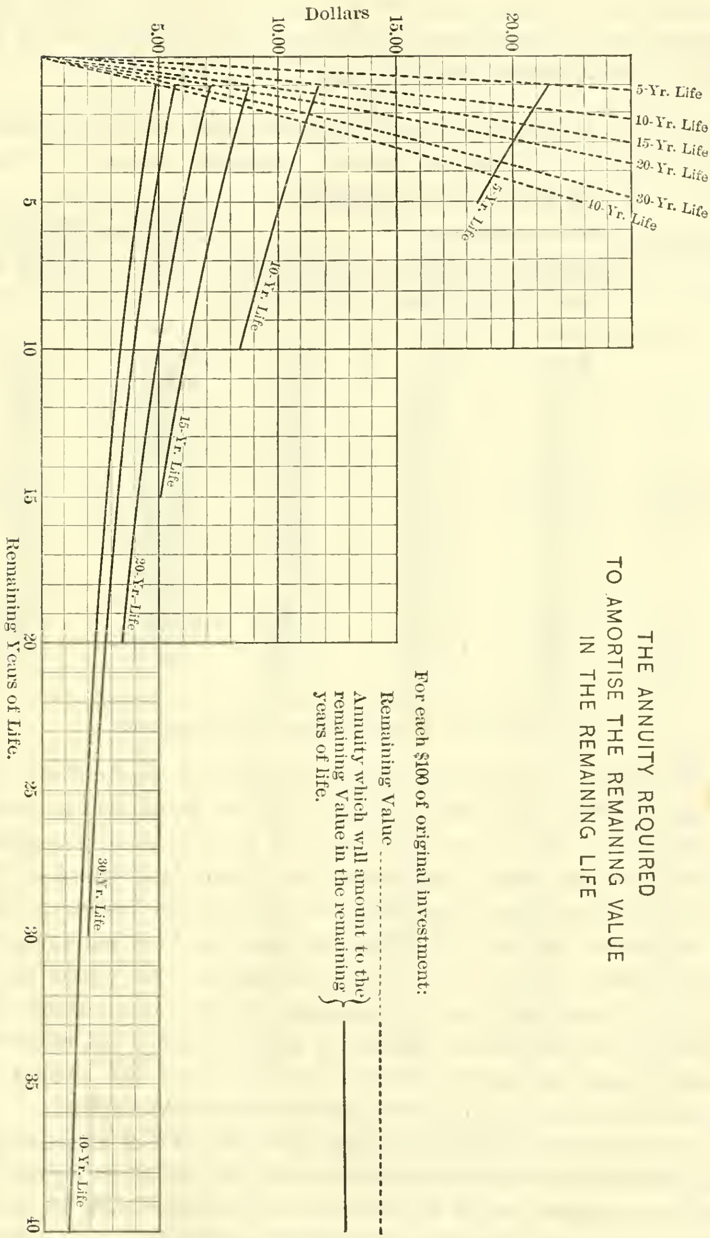
THE ANNUITY REQUIRED
TO AMORTISE THE REMAINING VALUE
IN THE REMAINING LIFE
For each $100 of original investment:
Remaining Value
Annuity which will amount to the remaining Value in the remaining years of life.
Dollars
5.00
10.00
15.00
20.00
5-Yr. Life
10-Yr. Life
15-Yr. Life
20-Yr. Life
30-Yr. Life
40-Yr. Life
5-Yr. Life
10-Yr. Life
15-Yr. Life
20-Yr. Life
30-Yr. Life
40-Yr. Life
5
10
15
20
25
30
35
40
Remaining Years of Life.

FIG. 2.

inviolable for the replacement of the plant at the end of its life. The owner reaps no benefit from it whatever, beyond holding it as the means for replacing a worn-out plant.

TABLE 4.—AMORTIZATION AND ANNUITIES. 20-YEAR EXPECTANCY.

At the end of year.	ANNUAL AMORTIZATION INCREMENT FOR EACH DOLLAR INVESTED = $0.0336.			Annuity which will amount to $1 in the remaining life.	* Amount of an annuity of $0.05 in remaining life.
	Amount in amortization fund.	Value remaining in the physical properties.	Amount of the annuity in the remaining life.		
	$0.000	$1.000	$1.000	$0.03358	$1.489
1	0.034	0.966	0.929	0.03614	1.384
2	0.069	0.931	0.861	0.03899	1.282
3	0.105	0.895	0.795	0.04220	1.185
4	0.143	0.857	0.733	0.04582	1.091
5	0.182	0.818	0.672	0.04994	1.001
6	0.223	0.777	0.614	0.05467	0.915
7	0.265	0.735	0.558	0.06014	0.831
8	0.309	0.690	0.505	0.06655	0.751
9	0.355	0.645	0.453	0.07415	0.674
10	0.403	0.597	0.403	0.08329	0.600
11	0.453	0.547	0.355	0.09449	0.529
12	0.505	0.495	0.309	0.10853	0.461
13	0.558	0.442	0.265	0.12661	0.395
14	0.614	0.386	0.222	0.15079	0.332
15	0.672	0.328	0.182	0.18463	0.271
16	0.733	0.267	0.143	0.23550	0.212
17	0.796	0.205	0.105	0.32036	0.156
18	0.861	0.139	0.069	0.49020	0.102
19	0.929	0.071	0.034	1.000	0.050
20	1.000	0.000	0.000	....	.....

* The annuity here noted is $1 divided by the expectancy.

The value of the plant in its varied stages of depreciation, plus the amortization fund, should at all times be equal to the capital invested in it. The owner, if he gets an annuity, as here assumed, is entitled at all times to the interest, not on a plant valued at first cost or investment less depreciation, but on the entire first cost. Had he determined, instead of building the plant, to keep his funds invested in safe securities at ordinary interest rates, he would, at the end of n years, have been in possession of his entire capital plus interest on the full amount thereof for the entire time. If, under the assumed facts, he were not allowed interest on the full amount invested in the public service plant, an injustice would be done.

This is true even when replacement takes the place of amortization. The owner in this case is entitled to interest on the entire capital invested in the plant, and, at the end of the plant's usefulness, he is also entitled to a return of the capital itself. Suppose that a city constructs a plant, paying cash for it, and collects rates which will just yield a fair rate of interest on the investment. At the end of n years the plant is replaced with a new one of the same capacity. As the city has not included in its rates theretofore charged an increment for amortization, it now finds itself in possession of a new plant and a total investment twice as great as the cost of the first plant. Applying the same principle to the second plant, rates should be doubled. This, of course, would be an absurdity. In the first instance they should have been fixed so that the remaining value of the plant, plus an actual or imaginary amortization fund, based on full expectancy (which may have been used in the meantime for other purposes), remains constant.

TABLE 5.—AMORTIZATION AND ANNUITIES. 30-YEAR EXPECTANCY.

At the end of year.	Annual Amortization Increment for Each Dollar Invested = $0.01783.			Annuity which will amount to $1 in the remaining life.	*Amount of an annuity of $0.033 ⅓ in remaining life.
	Amount in amortization fund.	Value remaining in the physical properties.	Amount of the annuity in the remaining life.		
	$0.000	$1.000	$1.000	$0.01783	$1.869
1	0.018	0.982	0.914	0.01888	1.765
2	0.056	0.964	0.890	0.02001	1.665
3	0.056	0.944	0.084	0.02124	1.569
4	0.076	0.024	0.790	0.02257	1.477
5	0.097	0.903	0.743	0.02401	1.388
6	0.118	0.882	0.697	0.02559	1.303
7	0.141	0.859	0.653	0.02731	1.221
8	0.164	0.836	0.611	0.02920	1.142
9	0.189	0.811	0.570	0.03134	1.066
10	0.214	0.786	0.531	0.03358	0.993
11	0.240	0.760	0.493	0.03614	0.923
12	0.269	0.731	0.456	0.03899	0.855
13	0.296	0.704	0.423	0.04220	0.790
14	0.326	0.674	0.389	0.04582	0.727
15	0.357	0.643	0.357	0.04994	0.667
16	0.389	0.611	0.326	0.05467	0.610
17	0.423	0.577	0.296	0.06014	0.554
18	0.456	0.544	0.269	0.06655	0.501
19	0.493	0.506	0.240	0.07415	0.449
20	0.531	0.469	0.214	0.08329	0.400
21	0.570	0.430	0.189	0.09449	0.353
22	0.611	0.389	0.164	0.10853	0.353
23	0.653	0.347	0.141	0.12661	0.263
24	0.697	0.303	0.118	0.15079	0.221
25	0.743	0.257	0.097	0.18463	0.181
26	0.790	0.210	0.076	0.23550	0.142
27	0.840	0.160	0.056	0.32036	0.104
28	0.890	0.110	0.036	0.49020	0.068
29	0.944	0.056	0.018	1.000	0.033
30	1.000	0.000	0.000	....	....

*The annuity here noted is $1 divided by the expectancy.

The same principle applied to a plant made up of a number of parts with various periods of expectancy will show that, in making appraisals for rate-fixing purposes, no reduction for depreciation should be made from capital actually and reasonably invested, provided, of course, that the amortization annuity is computed on the basis of full expectancy for each part.

TABLE 6.—AMORTIZATION AND ANNUITIES. 40-YEAR EXPECTANCY.

At the end of year.	Annual Amortization Increment for Each Dollar Invested = $0.01052.			Annuity which will amount to $1 in the remaining life.	*Amount of an annuity of $0.025 in the remaining life.
	Amount in amortization fund.	Value remaining in the physical properties.	Amount of the annuity in the remaining life.		
	$0.000	$1.000	$1.000	$0.01052	$2.376
1	0.011	0.989	0.951	0.01106	2.260
2	0.021	0.978	0.905	0.01163	2.149
3	0.033	0.967	0.860	0.01224	2.042
4	0.045	0.955	0.817	0.01289	1.940
5	0.057	0.943	0.775	0.01358	1.841
6	0.070	0.930	0.735	0.01435	1.746
7	0.083	0.917	0.697	0.01510	1.655
8	0.097	0.903	0.660	0.01593	1.567
9	0.112	0.888	0.624	0.01686	1.483
10	0.126	0.874	0.591	0.01783	1.402
11	0.142	0.858	0.557	0.01888	1.324
12	0.158	0.842	0.526	0.02001	1.249
13	0.175	0.825	0.495	0.02124	1.177
14	0.193	0.807	0.466	0.02257	1.108
15	0.211	0.789	0.438	0.02401	1.041
16	0.230	0.770	0.411	0.02559	0.977
17	0.249	0.751	0.385	0.02931	0.915
18	0.270	0.730	0.360	0.02920	0.856
19	0.291	0.709	0.336	0.03134	0.799
20	0.312	0.687	0.313	0.03358	0.744
21	0.336	0.664	0.291	0.03614	0.692
22	0.360	0.640	0.270	0.03899	0.641
23	0.385	0.615	0.249	0.04220	0.582
24	0.411	0.589	0.230	0.04582	0.546
25	0.438	0.562	0.211	0.04994	0.501
26	0.466	0.534	0.193	0.05467	0.457
27	0.495	0.505	0.175	0.06014	0.416
28	0.526	0.474	0.158	0.06755	0.376
29	0.557	0.443	0.142	0.07415	0.337
30	0.590	0.410	0.126	0.08329	0.300
31	0.624	0.376	0.112	0.09549	0.265
32	0.660	0.340	0.097	0.10853	0.230
33	0.697	0.303	0.083	0.12661	0.197
34	0.735	0.265	0.070	0.15079	0.166
35	0.775	0.225	0.057	0.18463	0.135
36	0.817	0.183	0.045	0.23550	0.106
37	0.860	0.140	0.033	0.32036	0.078
38	0.905	0.095	0.021	0.49020	0.051
39	0.051	0.049	0.011	1.000	0.025
40	1.000	0.000	0.000	....	......

* The annuity here noted is $1 divided by the expectancy.

In other words: Though eminently proper to deduct depreciation when determining the value of a plant for an owner or a purchaser, it is fundamentally wrong to make such deduction when the plant is being appraised for rate regulation, unless, as will be hereinafter explained, the amortization be computed thereafter on the basis of the remaining life of the plant or of its parts.

TABLE 7.—ANNUITIES WHICH WILL AMOUNT TO THE REMAINING VALUE OF PERISHABLE PROPERTY IN ITS REMAINING LIFE.

For each $100 of Original Investment; 4% Interest.

At end of year.	10-YEAR EXPECTANCY.		20-YEAR EXPECTANCY.		40-YEAR EXPECTANCY.	
	Remaining life.	Annuity.	Remaining life.	Annuity.	Remaining life.	Annuity.
	10	$8.33	20	$3.36	40	$1.05
1	9	8.66	19	3.49	39	1.09
2	8	9.01	18	3.63	38	1.14
3	7	9.37	17	3.78	37	1.18
4	6	9.75	16	3.93	36	1.23
5	5	10.13	15	4.08	35	1.28
6	4	10.54	14	4.25	34	1.34
7	3	10.96	13	4.42	33	1.39
8	2	11.40	12	4.60	32	1.44
9	1	11.85	11	4.78	31	1.50
10	..	.....	10	4.97	30	1.56
11	..	.....	9	5.17	29	1.62
12	..	.....	8	5.38	28	1.68
13	..	.....	7	5.19	27	1.75
14	..	.....	6	5.82	26	1.82
15	..	.....	5	6.05	25	1.89
16	..	.....	4	6.29	24	1.97
17	..	.....	3	6.54	23	2.05
18	..	.....	2	6.79	22	2.12
19	..	.....	1	7.08	21	2.22
20	..	.....	..	....	20	2.39
21	..	.....	..	....	19	2.40
22	..	.....	..	....	18	2.49
23	..	.....	..	....	17	2.59
24	..	.....	..	....	16	2.70
25	..	.....	..	....	15	2.81
26	..	.....	..	....	14	2.92
27	..	.....	..	....	13	3.03
28	..	.....	..	....	12	3.16
29	..	.....	..	....	11	3.28
30	..	.....	..	....	10	3.41
31	..	.....	..	....	9	3.55
32	..	.....	..	....	8	3.69
33	..	.....	..	....	7	3.84
34	..	.....	..	....	6	3.99
35	..	.....	..	....	5	4.15
36	..	.....	..	....	4	4.32
37	..	.....	..	....	3	4.49
38	..	.....	..	....	2	4.67
39	..	.....	..	....	1	4.86
40	..	.....	..	....		

This can best be made clear by an illustration: Let it be supposed that the passenger rates and the freight tariff on a steamboat line are subject to regulation, and that some one going into the steamboat business builds a steamer for the service. Let it be assumed, too, that in connection with this business he requires no capital investment other than the cost of the steamer, that terminal facilities, office space, and whatever else he needs are obtainable by rental. For the purpose of this illustration, let it be further assumed that the volume of business is such that there is no doubt about the income, so that the element of hazard is eliminated.

If, now, the steamboat has a life of 20 years, it will gradually depreciate in value and will go out of service at the end of a 20-year period. Ignoring its possible scrap value, which is immaterial for the purpose of this illustration, the following questions are to be considered.

At the end of 10 years, with interest at 4% per annum, and earnings just sufficient to yield interest plus an amortization, figured for a 20-year life at $0.03358 on each dollar of the investment:

1.—What will be the value of the steamboat to the owner at the end of 10 years?

2.—What will be the amount that a purchaser can afford to pay for the steamboat at the end of 10 years?

3.—What should the earnings be during the time the steamboat is in possession of the original owner?

4.—What should the earnings be during the time the steamboat is in the possession of a purchaser after 10 years of service?

The first and second questions have already been answered. The owner, by one line of reasoning, finds the remaining value in the steamboat to be 59.7%; the purchaser, by a different line of reasoning, finds the same value.

The third question, too, has already been answered. The original owner is entitled to a net return during the entire period of his ownership of 4% on his investment, which is at all times 100 per cent. No reduction is to be made for depreciation, because the fund which results from the accumulation of the amortization annuity, together with its interest, is available for no other purpose than the replacement of the steamboat at the end of its period of usefulness. It is dead capital, and remains dead until the property is disposed of or until required to replace the worn-out steamboat. The original owner, therefore, is entitled to a return of $4 + 3.358 = 7.358\%$ per annum on his investment.

In considering the fourth question, it may at first appear as though the purchaser, having invested only 59.7% could claim a return on this investment alone—that he should be allowed, in addition to the amortization as above determined, net earnings of $2.388 (4% on $59.70) per annum on what he paid for each $100 of the original cost of the steamboat; that the valuation for rate-fixing purposes, in other words, should be the original investment less depreciation. Under the adoption of this view, it will be seen that, if the steamboat were sold repeatedly, there would be a constantly decreasing appraisal for rate-fixing purposes.

In the last year of its service the valuation entitled to consideration in fixing earnings would be only 7 per cent. This view is unfair to the owner of the property, who should be assumed to be planning a continuation of the steamboat business. When he takes possession of the steamer, its value to him, as already set forth, is 59.7%, but, as owner, he at once finds that, of his capital ordinarily available for other purposes, an amount equal to 40.3% of the cost of a new steamboat is tied up in his steamboat business. It has become dead capital, for all purposes except replacement, as long as he remains in the steamboat business. This 40.3% at interest at 4% is necessary to supplement the annuity regularly going into the amortization fund, together with which at the end of the 20-year period it will just replace the steamer. Whether or not the 40.3% is actually set apart is immaterial; the fact remains that ownership of the depreciating steamer renders this amount of capital unavailable or dead for any purpose other than replacement, and the owner, no matter when he comes into possession, is entitled, therefore, to interest on this 40.3% just as fully as on the 59.7% which he paid for the steamer.

The demonstration of this fact may be made as follows: The purchaser of the steamboat, who buys the boat when it has a remaining period of usefulness of 10 years, invests, as has been explained, $59.70 for each $100 of the original cost of the steamboat. He is unquestionably entitled to interest on this sum, together with amortization, which at the assumed interest rate of 4% will be:

Interest at 4% on $59.70	$2.39
Amortization at 8.33% for the remaining 10 years, during which his investment is paid back to the purchaser	4.97
Total	$7.36

This is exactly the same as though, instead of the value of the steamboat, the capital originally invested had been taken into account, in which case the original owner or purchaser would be allowed:

Interest at 4% on the investment of $100	$4.00
Amortization annuity to retire $100 of the investment within the life of the steamboat, that is, within 20 years	3.36
Total	$7.36

Although it may be superfluous, one more illustration of this principle will be given: Let it be supposed that the owner borrows money from a bank at 4% per annum to build a steamboat, and that he earns 4% plus the amortization increment of 3.358 per cent.

Of the $7.358 to his credit at the end of each year's business for every $100 of capital invested, he pays the bank $3.358 on account of principal and so much of the remaining $4.00 as may be necessary to meet the interest then due. This will be all of the $4.00 the first year, and a decreasing amount thereafter until the end of the 20-year period, when his steamboat is retired. He then finds that he has already paid back to the bank on account of the borrowed capital twenty annuity increments of $3.358, amounting to $67.16, and that there is, therefore, still due to the bank $33.84. He also finds that the various amounts remaining in his hands from year to year, $0.134 at the end of the second year, $0.269 at the end of the third year, $0.336 at the end of the fourth year, and so on, together with interest thereon at 4%, when computed for the 20-year period will amount to the $33.84, the balance due at the bank. The owner finds he has earned nothing. He has made no investment and has received no return, which is as it should be in this hypothetical case. The rates, however, throughout the entire 20 years were fixed on the principle that 4% per annum should always be allowed on 100% of the capital invested, together with the amortization annuity, but without any deduction for depreciation. They could not have been fixed lower without entailing loss to the owner.

The value of a revenue producing property when the earnings thereof include an amortization annuity has already been discussed. It remains to consider the case of a property which, in addition to the accepted reasonable rate of interest (net), is earning a replacement increment determined by some formula, as above explained, instead of the annuity computed from amortization tables.

In this event, each part of a plant as it wears out is replaced out of current earnings. The owner does not maintain an amortization fund, neither is any of his capital rendered dead or unavailable. To him the value of the property is at all times 100%; so, too, in the case of a purchaser. Knowing that the replacement is covered fully in the earnings, he is willing to pay 100% for the plant, regardless of its depreciation.

Take again the case of the steamboat with a life of 20 years. On the assumption that the replacement cost of the steamboat will be returned to him when the steamboat is worn out, a purchaser will pay for it at any time in its life 100 per cent. Of course, in the case of a single steamboat, it might be regarded as unreasonable to assume that in one or more remaining years of its usefulness it will earn enough in excess of reasonable interest on capital invested to pay for a new boat; but if, instead of one steamboat, there were twenty in use, and the annual replacement increment were one-twentieth of the invested capital, or one steamboat each year, then, without hesitation, the purchaser would value the property at 100 per cent.

When, therefore, the actual average annual replacement increment can be earned in excess of a reasonable interest on the invested capital, then the appraisal for an owner, for a purchaser, and for rate-fixing purposes, would be uniformly and always 100% of the capital invested. The term, "value," in this case, means the same to the original owner, to the purchaser, and to the ratepayer.

For rate-fixing purposes, the steamboat, or the business which the steamboat represents, is to be valued throughout the entire period of the steamboat's usefulness at 100%; and the earnings, when amortization is included, should be $4 + 3.358 = 7.358\%$ on this valuation.

Another case has already been considered. Suppose that, preceding the time of an appraisal for rate-fixing purposes, earnings have been inadequate to supply any amortization increment, and that it be determined thereafter to allow the actual annual replacement requirement to be earned. What, in this case, should be the appraisal?

The original investment being 100%, there having been no amortization annuity in the past, there can be no transfer of the property at less than 100% without loss; but if, by reason of inadequate returns, the market value could not be maintained at 100%, and a sale has been made at less than this sum, the new owner will be compensated and protected if, on his investment, which is not original cost, he earns reasonable interest and an adequate amount for replacements. This must be so, because, in the future, actual replacement requirements being covered by the earnings, the worn-out parts will be replaced without cost to the owner. This replacement neither increases nor decreases his investment; but, if the property is extended and new parts are added, such additions represent newly invested capital to the full amount of their cost, and in such a case his investment, expressed as a percentage of the total cost, will gradually increase.

At all times, however, without causing loss to the new owner, that part of the plant which he bought at a depreciated value could be valued at his purchase price, while all extensions subsequent to the purchase should, for rate-fixing purposes, be appraised at 100 per cent. Such a course, however, would deprive the new owner of the opportunity for profit, of which he probably thought to avail himself when he bought a plant of depreciated value, and would place the rate-payer in the position of having made a profit at the expense of the original owner. This fact, however, explains why the market value of stocks and bonds is cited so frequently as an indication of value.

It may be held that a determination of value for rate-fixing purposes, on the principles herein set forth, is not a determination of value at all. This may be true, but it then becomes a matter of defining "value," and a distinction should be made between value and the appraisal of the investment on which rates may be properly based.

The term, "value," has been very generally used in matters involving the fixing of rates in the past. Perhaps when the facts herein set forth are better understood, more attention will be paid to the capital reasonably and properly invested.

The illustration with a steamboat which, though subject to constant depreciation in value, is rendering the same adequate service throughout its entire period of usefulness, was selected because thereby the fundamental principle involved is made plain. This principle is much less apparent when a plant made up of many parts of various ages and of various periods of usefulness is under consideration. For example, a plant more than 40 years old, of gradual growth, all parts of which have a life of 40 years, would have a selling value of 63.80%, if the proper provision for amortization, based on full expectancy, has been made; but, in such case, it should earn reasonable interest on 100% of its cost.

A plant more than 20 years old, made up of many elements or parts, all having a useful life of 20 years, but constructed one-twentieth each year, should be worth 58.95% to a purchaser, but, with provision for amortization, as above, should earn a reasonable interest on 100% of its cost.

A plant more than 5 years old, all parts of which have a life of

5 years, constructed one-fifth each year, should be worth 61.58% of cost to a purchaser, but when the allowance for amortization is based on full expectancy, a reasonable interest should be earned in addition thereto on 100% of the investment.

It follows from the foregoing, not only that for rate-fixing the appraisal may properly be of the capital invested, but that, in determining this capital, the aggregate replacement cost, within periods not greater than the expectancy of the several perishable parts of a public service plant, may have to be taken into account.

The amount which should be returned to the owner as a replacement allowance is the capital actually invested in the part of the plant replaced from time to time. It is not the original cost, but the cost at the last renewal, which is to be returned to him, and which he is expected to re-invest with such addition thereto or subtraction therefrom as changed conditions may compel.

The account, as far as a discarded appliance is concerned, is closed, and the new appliance which takes its place, in fact represents new investment; and in its appraisal no note whatever is to be taken of the conditions under which its predecessor was constructed, or installed.

The appraisal of capital invested, therefore, should deal with conditions as they have prevailed during a longer or shorter period antedating the time of the appraisal. When a complex plant is under consideration, prices used in estimating cost should be average prices and not those prevailing at any particular time.

Under a system of permitting the owner of public service properties to earn from year to year the actual average replacement requirements, the necessity for a close distinction between repair and replacement disappears. This is of some advantage, as it is at best difficult to discriminate between small items of replacement and large repair items.

By the foregoing reasoning the conclusion seems inevitable that there may be cases in which large public service properties, such as sewer systems, harbors, railroads, and the like, the ownership of which is not limited in time by franchise, may be regarded as more or less complex plants, having practically perpetual life. The appraisal for rate-fixing purposes is then at the full amount of capital reasonably and properly invested, and there will be no deduction therefrom for depreciation. There will be no amortization if constructed on a cash basis, and all repair and replacement requirements will then appear in the expense of operation and maintenance, but with due regard to all the elements that should be taken into account.

Real estate is usually considered as requiring no allowance for depreciation, because, as a rule, real estate does not depreciate in value. However, cases are conceivable where there is depreciation, where, possibly by reason of the advance in the arts and abandonment of certain properties, the encumbered ground on which useless improvements are located may have less value than its original cost.

In such cases, if they could be foreseen, there might well be some allowance for depreciation. Ordinarily, however, there is a gradual increase in the value of real estate. This increase, strictly speaking, should be regarded as earnings, a point to which reference will be made later. As a rule, the present value of real estate, in lieu of its first cost plus such improvements as grading, bulkheading, reclaiming against submersion, street and sewer work, and the like, may be entered on the appraisal. As the present value can generally be readily ascertained, this is usually adopted as a sufficiently close approximation of capital invested in real estate.

Intangible Values.

Ordinarily, there is neither occasion for nor propriety in adding, to an appraisal for rate-fixing purposes of a public service property, anything for intangible values, such as franchise, going concern, and the like. When an addition to the appraisal for these is made, it is most likely for the purpose of giving a name to an addition which is necessarily more or less arbitrary.

This statement, of course, does not apply when the State or a municipal authority has been paid for a franchise.* The cost of the franchise, in such a case, is a part of the legitimate investment of capital, and must be included in the appraisal. The same is true of water rights. Where adverse rights have to be quitted, or where, as under a new law in California, the State makes a charge for water rights, their cost is a legitimate expenditure, and should not be classed among the intangible values in the sense in which the term is here used.

Neither does the foregoing statement relating to intangible values

* In San Francisco, for example, franchises for street car lines are sold to the highest bidder.

apply when the appraisal is made of a property having a definite earning capacity. The sum of all intangible values is then determined by capitalization of net earnings and by deducting from such capitalization the valuation of the physical properties.

When rates are being fixed, it is quite proper to allow earnings in excess of earnings on ordinary safe investment. Such allowance may be made either direct, as an addition to the allowed rate of interest, or in the roundabout way of an addition to the appraisal.

It is possible, of course, in the case of large earnings in the past, that a portion thereof should be considered as capital returned to the owner. In such a case the fact may be of some importance that an appraisal at 100% of the investment would already include some of the intangible value.

When the annual amortization increment has not been fully covered by the earnings, the deficiency is a loss. This can be made good to the owner only by increasing the earnings, which, as previously stated, is sometimes done by computing the interest to be earned, not on the invested capital alone, but on the investment plus the aggregate deficiency in the earnings of past years. Such deficiency of earnings, however, can hardly be regarded as an element of value.

Intangible values, of whatsoever nature, result from high earnings. In the case of public service corporations, they are arbitrarily created by agreeing to, and permitting, rates which produce a revenue in excess of the ordinary return on safe investments. They do not exist unless the rates are higher than those which would produce net earnings equalling an ordinary interest return on the properly invested capital.

It is eminently proper to treat expenditures such as the interest on capital during construction as an item of cost; yet the propriety of doing this is sometimes questioned. An inadequate interest return during the development stage is another matter. Expenditures may be incurred which can be classed as development expense, such as advertisements and the salaries of business solicitors, but these are ordinarily and with perfect propriety classed as general expense, or are otherwise included in the operating expense, and enter into consideration when net earnings are estimated. In other words, they should be repaid from year to year as they are incurred, and should not be considered as a part of the capital on which the owner is entitled to a return.

In some cases, it may be possible to segregate such expenditures and to determine, too, whether they, together with the aggregate loss of interest during the unproductive period in the history of a plant or of parts of a plant, have already been made good by high rates in the past. If this is found to be the case, the element of hazard is to a large extent eliminated.

The public service corporations, naturally, would prefer to have the losses during the lean years, and such expenditures as the advertising of the business, classed as investment of capital. The apparent investment is thereby increased and the apparent aggregate profits of the business figured from the beginning of the operations are thereby made to appear larger than they would otherwise.

The fact that interest during construction is properly considered a part of cost is, as a matter of course, as true of all work of extension and replacement as it is of original construction.

Where, as in California, the water companies and the gas companies operate under constitutional privileges, without special franchise, the hazard of the business should be covered in the earnings, and these earnings should amortize, in the course of time, a reasonable allowance for inadequate earnings, or other unavoidable losses of past years. This, of course, can be done by making an arbitrary addition to an appraisal, but then, as already stated, the hazard of the business is thrown in large measure on the ratepayer, and the rate of return must be relatively low. It is quite as effective to keep the appraisal low and make the rate of return relatively high.

There seems to be no question that the part of value usually designated as "going concern" is intended to apply to the advantage which an established business has over a corresponding prospective business, foreseen as the result of investment, but not yet established.

As long as the business is unprofitable, and as long as the rates charged do not yield a net return on the invested capital which exceeds the return obtainable from savings banks or from other investments of a character regarded as safe in the ordinary acceptance of this term, the business has no "going concern" value. This value, like franchise value, can result only from a capitalization of the excess of net earnings over the return on ordinary safe investments. It is generally a purely fictitious value, without basis other than that which results from high net earnings, but may be, and often is, regarded and defined as that portion of the intangible value for which some sort of a demonstration can be offered, as, for example, the equity of making good early losses and the deficient earnings of the past, or some estimated cost of establishing the business at the time of the appraisal, including loss of interest during an assumed time which would be required for reconstruction. It is held, with some reason, that in equity it is proper to assume that, if the community which is served by a public service property had undertaken construction and management itself, it would have subjected itself to the same losses, or at any rate to the same chances of loss, as the owner of the property, who is in some measure at least to be regarded as agent and who, as such agent, should neither be made to suffer unavoidable losses nor yet be allowed to make unreasonable profits.

The special franchise, when one exists, defines the limits within which an owner must operate. If it does not permit rates which will make the net earnings adequate, then the losses must fall without recourse on the owner; if, on the other hand, the net earnings are greater than the returns on safe investments, then, with due regard for the time during which the rates are protected by the franchise, these earnings are the basis from which, with a fair degree of precision, the aggregate amount of intangible values may be determined. These values collectively must be the difference between the capitalization of the total net earnings (properly determined) and the capital which remains, at any particular time, as an investment in the property.

The early losses and deficient earnings, when they are added to the valuation, are regarded by the appraiser as a part of the investment which had to be made to get the business going—to establish it—or at any rate to carry it along until it was on a paying basis. If this procedure should be generally accepted, it would result in giving to "going concern" the greatest value in those cases where, at the outset of the undertaking, conditions were the most unfavorable. This is an absurdity, because the valuation should be a valuation under present-day conditions, and the actual advantage which an established business has over another that would result from a duplication of the plant may be, and generally is, entirely independent of the conditions which prevailed when the established business was in its infancy.

It may be, of course, and sometimes has been held, that, if unsuccessful work and early losses are not to be added to the cost of a property, interest during construction likewise should not be treated as cost; but, in one case, there is no limit to the possible amount of unproductive expenditure, while, in the other, a definite assumption applicable in practically all cases can be made. It is not unfair to assume, for example, that in case of water- or gas-works of mature age and gradual development, some period of time, most naturally for small investments one year, will cover the average time before they commence to be remunerative. Where large and complex works are under consideration, the cost for one-half of the period of construction may be a fair allowance. The amounts thus determined are incident to every construction, whether new or whether in the nature of a hypothetical replacement, and, therefore, with perfect propriety, may be added to cost. It is not so with the expenditures of uncertain and extremely variable amount which may be made for unsuccessful work. There may be none in one case, while in another they may be very large, as, for example, in the case of the failure of an expensive structure like a dam.

While the early losses and the expenditures for unsuccessful work are not a measure of going concern value, they are nevertheless of that class of expenditures which, in whole or in part, as already stated, should come back to the owner of the property sooner or later. To add them in the exact amount shown by the cost records in any particular case is not an invariably fair procedure. The owner who builds with care and under the best expert advice and has no such losses is entitled to a reward for his good judgment and for the care with which he has executed the works. The "going concern" value of such works is certainly as great as the going concern value of other works of a similar character and extent which, by reason, perhaps, of less care in design and execution, involved a large expenditure for unsuccessful work and for the development of the business.

The combined experience on all works of a similar character, however, should, in the long run, establish the addition which should in fairness be made to the earnings to amortize an assumed fair allowance for this class of expenditures within a reasonable and not too short time. This addition may be relatively large for one type of works and small for another. It seems fair to assume that it should be relatively small when the total values are high.

Occasionally, a definite basis for at least a part of the value as an established business can be found. For example, there are cases in which the cost of making a connection with a water or gas main is a charge in whole or in part against the consumer. In such cases the cost of making the connection is no part of the capital invested by the owner of the water- or gas-works, and should not be included in an appraisal of the physical properties; but, to the extent of the cost of renewing the connections with a new system of mains, the established company has a distinct and easily recognized advantage over any new company. While not to be taken into account at all in making appraisals for rate-fixing purposes, it may, when intangible values under special franchises are to be determined, be regarded as a part of the aggregate intangible value obtained by capitalizing the excess of the earnings over the ordinary return on safe investments not involving management.

It appears from the foregoing that, no matter how accurately the aggregate of the intangible values may be determined, it is frequently impossible to find any other than an arbitrary basis for separating them into such subdivisions as "going concern," "development of business," "franchise," "unification of system," and the like. Fortunately, such separation is rarely necessary, and, when attempted, is usually only for the purpose of giving a reason why an arbitrary allowance of earnings above those on ordinary safe investments is just and proper.

When the losses during lean years, or deficient earnings, or unproductive expenditures, such as water tunnels or wells which produce no water, structures that fail during erection, damage by fire, flood, earthquake, or explosions during erection, are added to the value as "going concern," this is unnecessary and forced. These, as has been stated, are losses, and, therefore, are to be considered and treated as the reverse of earnings. They cannot in all cases with propriety be added to the valuation of the physical properties, though it may be eminently proper, on account of such originally unforeseen circumstances, to estimate the cost of reproduction liberally.

In some form they should be taken into consideration in fixing rates. It is rarely practical to determine such losses with accuracy, and yet it is well known that very few public service plants commence operation without some untoward experience or without being compelled to do business for a time at a loss. Frequently, the expenditures for unproductive work are large, and yet this unproductive work should ordinarily be assumed to have been done under competent advice. It is assumed, in other words, that it could not be foreseen that what turned out to be unproductive work would have no value.

The easy way out of the trouble of providing compensation for such expenditures is the one frequently recommended, to add them to the valuation, giving them a name and treating them as a part of the intangible value; but, while this may appear reasonable in ordinary cases, where the expenditure for useless work and the losses in lean years have been small, other cases have occurred and can be foreseen, as already explained, in which the problem will not be as easy of solution. It is never logical.

Where there has been loss due to some unforeseen condition, due perhaps in part to error of judgment and to lack of proper foresight by the owner, it is eminently proper to let a part of this loss fall on the owner. When he embarks upon the enterprise he must be supposed to do so with the fixed purpose of reaping a profit:

1st.—In the high rates which the people practically guarantee to the owner;

2d.—In the advance in real estate and other values which make up the business.

If, now, such anticipated increase in value is allowed to the owner and the rates are fixed with a view to covering the ordinary hazards of installation and operation, and to provide proper compensation for management, then the owner on his part must stand, in part at least, the unforeseen losses, such as the destruction by flood of a partly finished dam, in the assurance that in the long run these losses will be made good, as far as they ought to be made good, by adequate compensation for the service which he renders.

It follows that all intangible values (as they may come into consideration apart from appraisals for rate-fixing purposes) should result from the inclusion of some more or less arbitrary allowance in the established rates such that earnings will exceed in some predetermined measure the earnings which would just yield the ordinary interest rate on safe investments when applied to the reproduction cost of the plant, or better yet, when applied to the actual capital reasonably and properly invested. When, for any purpose, consideration is given to intangible values thus determined, it will matter but little what name is used to designate them. It becomes comparatively easy, too, in such a case, to establish a proper relation between the tangible and intangible values, such that both owner and ratepayer may receive equitable treatment.

If it is proper to add anything for early losses, unproductive investments, and cost of developing business to an appraisal, then it is equally proper, in fairness to the ratepayer, to exclude from the appraisal all accessions of value, all appreciations which result from advance in the value of real estate and like causes, and it will also be fair and proper to keep the net earnings at and not above the ordinary return on safe investments.

When the cost of unproductive work, as just referred to, is added to the capitalization, it is with the idea that this addition shall be treated for all time as a part of the investment, and not as a loss, and that the ratepayer must bear the additional burden for all time.

When the cost of useless elements or early losses in the business are treated as losses, they should in a fair measure be made good in the course of time out of adequate earnings, and this should be done irrespective of whether every item of early loss or of every unprofitable investment can be remembered or not.

The most logical course to be pursued, and the one which is always open to the appraiser, is to use the best available means for determining the amount of capital which is properly invested, then determine what the earnings should be to yield an ordinary return on the investment thus ascertained, and then to increase those earnings by an arbitrary amount, which may vary within wide limits, not only to compensate for past losses and for the hazard during construction and operation, but also as a compensation for management.

In doing this, however, every endeavor should be made to determine correctly the cost of operation and maintenance. Maintenance is here used in its broadest sense, and must include amortization. Care must be taken, also, not to confound amortization with depreciation, because, as has been explained, an amortization annuity, figured at compound interest, is not available to retire invested capital until at the end of the life of a plant, and the existence of an amortization fund is not in itself a reason for decreasing the capital allowance on which interest is to be earned.

Fundamental Principles.

1.—The valuation of a public service property and its earnings must bear such relation to each other that there will be returned to the owner, within the life of the property, the capital which he has properly invested in it, and in addition thereto, interest at a reasonable rate, upon such amount of capital as from time to time actually and properly remains in the property as an investment.

2.—Amortization by the annuity method (the amortization or depreciation annuity being based on the full expectancy) is amortization at the end and only at the end of the period embraced in the expectancy. The invested capital remains uniform throughout the entire period.

3.—In the case of amortization by the annuity method, the value of a plant as it would be determined for a purchaser is the cost of replacement (or original investment) less the amount of the amortization or depreciation annuity at the time of purchase.

4.—Amortization by the straight-line, or direct percentage, method is amortization in annual installments. The invested capital is reduced from year to year.

5.—In the case of amortization by the straight-line, or direct percentage, method, value and the appraisal for rate-fixing purposes are determined in the same way.

6.—When the annual earnings are just adequate to meet operating expenses, interest, and the annual replacement, the amount set apart for replacements will not reduce the invested capital.

7.—A public service property which consists of a single perishable item may, at any time of its life, be appraised at 100% of the capital properly invested, provided that amortization, estimated by the annuity method for the full expectancy, has been allowed from the beginning.

8.—A public service property which consists of a single perishable item may, at any time of its life, be appraised at the investment less depreciation (determined by any method), and amortization may then be computed for the remaining value thus determined, but must be based on the remaining years of the property's usefulness.

9.—A public service property made up of numerous items, all of which have the same expectancy, may be appraised at 100% of the investment, and amortization should then be allowed from the beginning, and the full expectancy should be used in computing it.

10.—A public service property made up of numerous items, all of which have the same expectancy, may have each item valued separately, as under Paragraph 8, with deduction for depreciation, and with amortization allowed for the remaining life of each item.

11.—A public service property, of gradual growth and mature age, made up of numerous items of the same expectancy, when the assumption is justified that the annual rate of extension has been uniform, may be appraised at investment less an average depreciation, and amortization is then to be allowed for the equivalent remaining life of an equivalent single item.

12.—When a public service property is made up of many items of various expectancies, the property may be appraised at 100% of the investment, and, amortization being allowed from the beginning, this is to be estimated on the basis of the full expectancy of each group of items of equal expectancy.

13.—When a public service property is made up of many items of various expectancies, each item may be dealt with separately, as under Paragraph 8, or groups of items may be dealt with, as under Paragraph 11.

14.—When the special case is presented in which there has been no amortization earned in the past, it will be proper to substitute the annual actual replacement requirement in lieu of amortization. The appraisal should then be at 100% of the capital properly invested.

15.—When the amortization annuity is based on the full expectancy and remains at this amount throughout the life of a plant, then no part of the amortization can be applied to retire the investment until the close of the period of useful life, when the amortization fund will be equal to the investment. In case it be thus applied, a new amortization rate for the remaining life and the remaining value must be introduced into the calculation. Table 7 gives such rates for a few expectancies.

16.—When the appraisal for rate-fixing purposes is investment less depreciation, and earnings have not included amortization in the past, then, under amortization computed by the annuity method for the full original expectancy, the owner will be operating at a loss.

17.—Proper investments for franchises, for water rights, and the like, are always to be included in the appraisal.

18.—Intangible values should be disregarded, in making appraisals for rate-fixing purposes, excepting only when the rate of net return is deliberately fixed at or too near the rate earned by ordinary safe investments, in which case an arbitrary addition to the appraisal, under whatever name, should be made. The interest on this item of the appraisal will be the reward of the owner for management and for any hazard which the business may involve.

19.—The net earnings of a public service property should in some measure exceed the return from ordinary safe investments.

20.—The appraisal of real estate should be at its present value.

21.—When the increase of the value of a public service property, due to increase in the value of real estate or like causes, is determinable in advance, such increase may be taken into account as a part of the current earnings.

22.—When, in the past, there has been increase of value, due to increase in value of real estate or like causes, this is to be offset against losses during lean years. The increase in value represents reinvested earnings.

It is to be noted, as set forth in Paragraphs 8, 10, 11, and 13, that a valuation for rate-fixing purposes at less than the original investment of capital may be perfectly proper. It represents the remaining investment; but, when the original investment less amortization or depreciation is introduced into the calculation, amortization requires special consideration, because it must be entered at a new and increasing amount from year to year. Reference may be had to Table 7, which makes this point clear.

Notwithstanding the great disadvantage attendant upon valuation at original investment less depreciation, such valuations are being made and are therefore being herein fully considered.

In explanation of the statement that the earnings of a public service property should be somewhat greater than those of ordinary safe investments, reference may again be had to the case of an owner of a public service property who invests only borrowed money. If he receives only such interest on the investment as he must pay to the bank, he will have rendered a service without compensation, except such as he may be allowed in salaries, under operating expenses. In such case, it would be a proper business arrangement to compensate him for the risk of loss which he assumes, and for his management, and to make this compensation in some measure proportional to the net earnings. If the owner in the cited case is a stock company, this compensation will be the only element giving value to the capital stock of the company.

The Appraisal at Cost of Reproduction.

The objection may be made that, in the practical application of these principles, the capital properly invested cannot always be determined with sufficient accuracy.

It is reasonable to expect that, under good and intelligent direction, and competent expert advice, every dollar invested in a public service property will have been properly expended. Under less able management, there may be a waste of capital, and the works, when completed, will then have cost more than they should. The book accounts, therefore, cannot be accepted as conclusive evidence, even when it can be shown that the cost account has been properly kept. What is wanted is a method or plan of valuation which can be applied under all circumstances in perfect fairness to both the owner of a property and to the ratepayer. There appears to be none better than that of estimating the capital, properly invested, by an appraisal of the public service property at cost of reproduction, item for item, using, however, as a basis for appraisal, not the prices of labor and material which prevail on any particular day, but the prices which represent averages for some considerable time in the past.

Under this method of appraisal, which is recommended as fair in estimating capital reasonably and properly invested, only properties in use are to be included in the appraisal, and under it the owner who has built with intelligence and economy finds himself liberally treated, while the owner who has built wastefully and has incurred useless expenditures is made to bear the penalty of his wastefulness.

Increase in value not represented by a direct investment of capital, as in the case of an appreciation of the value of real estate, may properly be regarded in the light of earnings when regulating rates. Such appreciation of value may also result from other causes, as in the case of an advance in the prices of material and labor, which would make the reproduction of a plant cost more than has actually been invested in it. On the other hand, there may be a decrease in value due to reduced prices of material or labor and the like. These changes are generally gradual and, when treated as income, or as expense, and distributed over a series of years, usually affect the general result but slightly.

In many cases, not only the increase in the value of real estate is small, but also the proportion of its value to the entire value of a property. In such cases, if there is uncertainty about first cost plus the cost of improvements, such as grading, filling, bulkheading, street and sewer work, the error made in ignoring the effect of a change in the value of the real estate will be small. A doubling of value in 40 years, for example, is equivalent to a rate of increase of 0.52% per year of the value at the end of the 40-year period. A doubling of value in 20 years is equivalent to a rate of increase of 1.68% per year of the value at the end of the 20-year period.

These percentages, if the real estate represents 10% of the total appraisal, would appear in the earnings as 0.17% and 0.05% per annum of the total appraisal; but, when appreciation of value is treated as earnings, then that portion of the earnings available for distribution is less than it would otherwise be, and the appreciation becomes in fact a reinvestment of earnings, and should be properly taken into account in making an appraisal of invested capital.

Thus, in the case of a property which has appreciated in value 100% in 40 years, if this appreciation has been the same in amount each year, and if it could have been determined in advance, there would have been entered into the calculation earnings by appreciation gradually decreasing as the property increased in value from 1.05 to 0.52% per annum. The rate of interest to be earned and distributed would have been decreased by these amounts. An appraisal at any time would then have taken the properties into account at their appreciated value.

In practical application of such a principle, difficulty arises in determining what allowance to make for the possible annual appreciation. No general rule can be laid down for this determination. It will probably be found that in most cases, in view of the small rate of appreciation, offset as it may be by losses and by depreciation not otherwise taken into account, this appreciation should go to the owner of the property as a more or less indeterminate part of the profit to which he is entitled.

Unless, therefore, there is good reason for taking into account the appreciation or the depreciation in the value of real estate as an addition to or a deduction from earnings, this element may be neglected. This is also true of all that portion of the plant which has increased in value by reason of an advance in the cost of labor and materials, in case the appraisal is based on the estimated cost of reproduction, as explained, because, in that event, the appraisal, being based on prices as they have prevailed during considerable time periods, will ordinarily show only moderate and gradual changes of value.

Disadvantage of Annual Rate Regulation.

In California the law requires that the water rates, to be charged by public service corporations which supply water to the inhabitants of cities and towns, shall be fixed annually by the proper authority. This requirement does not make for efficient service. It would be better, both for the owner of the public service property and for the ratepayer, to have rates regulated with less frequency. A 5-year interval would probably be about right. The certainty that an acceptable rate will prevail for at least a 5-year period would be an inducement to the public service corporations to render satisfactory service. Extensions would be made more willingly, and the needs of the ratepayer would be more likely to receive proper consideration than under the prevailing system of annual rate regulation, which involves in constant uncertainty business relating to the immediate future. The owner of the plant, knowing that each year his profit may be cut off by an inadequate rate limit, hesitates to make any investment beyond what may be imperatively demanded, with the result that the service becomes unsatisfactory or inadequate.

The Appraisal of the Investment.

It has been made clear in the foregoing that a valuation of the purely physical elements of a public service property (depreciation deducted), coupled with an allowance of amortization computed for the full expectancy, as is frequently done, would be inadequate as a basis for rate regulation. This fact is generally recognized by engineers who are called on to make appraisals for such purposes, and no doubt the amounts added as intangible values are sometimes intended to make good such deficiency, at least in part, even when the appraiser does not know why his appraisal is inadequate.

The necessity in such cases for the addition of something to the value of the purely physical elements of a public service property undoubtedly exists; but on the method of determining the amount of the addition, there has not heretofore been agreement. This is due to the imperfect analysis which has been made of such investments, from the business man's standpoint, and to the ruling of the Courts, which hold that owners of public service properties are entitled to a fair return on the "value" of such properties.

If it be found that the ruling of the Courts is not subject to modification, or, in other words, that appraisals must be "value," as "value" would be determined by a purchaser, that is to say, for the tangible elements in most cases, cost or cost of replacement less depreciation, or something practically equivalent thereto, then the appraisers making the valuation, who adhere to the method of computing amortization on the full expectancy, will be constrained to find intangible values in one form or another which will swell the appraisal to where they would like to see it for rate-regulation purposes, that is, about, or somewhat above, the amount of capital reasonably and properly invested.

Of course, a special franchise granting excessive returns is out of consideration in this statement. In such a case, the intangible values are real values determinable by a capitalization of earnings and a subtraction of the value of the tangible parts of the property.

It follows, too, that ordinarily it makes very little difference whether the intangible value is called "going concern," or "franchise," nor how it is arrived at, nor in what proportion it is apportioned to these two classes of value, nor whether a part thereof be otherwise designated, as for example, "adaptation and solidification of roadbed," as was done in a recent valuation of the railroads in Minnesota.

After all has been said, it will be found true that the adoption of the method of valuation for rate-regulation purposes at the investment without deducting depreciation (as herein advocated) will be always applicable, and, if properly applied, will protect the interests of both owner and ratepayer. It has a distinct advantage over other methods, which are involved in more or less obscurity and cannot be standardized. It will be resisted by certain corporations, the values of whose properties, based on earning power, have been greatly inflated, because thereby the facts showing the relation between net earnings and the capital properly invested in any enterprise are made apparent. It will be welcomed by the ratepayer and by all boards or commissions charged with regulating rates, and if generally adopted, will lead ultimately to a careful analysis of earnings by all owners of public service properties, in order that actual net earnings may be determined correctly. The relation of net earnings to the properly invested capital will always remain the most important factor to be weighed, when rates are to be regulated.

The excess of these earnings over the earnings which would represent a return on ordinary safe investments are the reward which the owner receives, as has been stated, for his management of the property and for assuming risks. By reason of the fact that the replacement or amortization requirement is necessarily more or less conjectural, the prospective net earnings cannot ordinarily be estimated closely. This is an additional reason why the rate of return should be made liberal. Any addition to the rate of return is then a purely arbitrary addition, and this addition capitalized, if there is certainty that it will be earned, is the real basis for the intangible values as they would be taken into consideration by a purchaser.

Of course, the proceeding can be reversed, and an arbitrary addition can be made to the appraisal, to which the rate of return is then applied in estimating what the earnings should be. It makes no difference, in the ultimate result, at which end the addition is made, and the appraiser in this matter may follow his own inclination.

Recent Court Decisions.

The United States Supreme Court, in Knoxville *vs.* Knoxville Water Company,* says:

"The first fact essential to the conclusion of the court below is the valuation of the property devoted to the public uses, upon which the company is entitled to earn a return. That valuation ($608,000) must now be considered. It was made up by adding to the appraisement, in minute detail of all the tangible property, the sum of $10,000 for 'organization, promotion, etc.,' and $60,000 for 'going concern.' The latter sum we understand to be an expression of the added value of the plant as a whole over the sum of the values of its component

* United States Reports, Vol. 212, p. 9.

parts, which is attached to it because it is in active and successful operation and earning a return. We express no opinion as to the propriety of including these two items in the valuation of the plant, for the purpose for which it is valued in this case, but leave that question to be considered when it necessarily arises. We assume, without deciding, that these items were properly added in this case. The value of the tangible property found by the master is, of course, $608,000 lessened by $70,000, the value attributed to the intangible property, making $538,000. This valuation was determined by the master by ascertaining what it would cost, at the date of the ordinance, to reproduce the existing plant as a new plant. The cost of reproduction is one way of ascertaining the present value of a plant like that of a water company, but that test would lead to obviously incorrect results, if the cost of reproduction is not diminished by the depreciation which has come from age and use.

*"The cost of reproduction is not always a fair measure of the present value of a plant which has been in use for many years. The items composing the plant depreciate in value from year to year in a varying degree. Some pieces of property, like real estate, for instance, depreciate not at all, and sometimes, on the other hand, appreciate. But the reservoirs, the mains, the service pipes, structures upon real estate, stand-pipes, pumps, boilers, meters, tools, and appliances of every kind, begin to depreciate with more or less rapidity from the moment of their first use. It is not easy to fix at any given time the amount of depreciation of a plant whose component parts are of different ages with different expectations of life. But it is clear that some substantial allowance for depreciation ought to have been made in this case."

* * * * * * *

†"A water plant, with all its additions, begins to depreciate in value from the moment of its use. Before coming to the question of profit at all, the company is entitled to earn a sufficient sum annually to provide not only for current repairs but for making good the depreciation and replacing the parts of the property when they come to the end of their life. The company is not bound to see its property gradually waste, without making provision out of earnings for its replacement. It is entitled to see that from earnings the value of the property invested is kept unimpaired, so that at the end of any given term of years the original investment remains as it was at the beginning. It is not only the right of the company to make such a provision, but it is its duty to its bond and stockholders, and, in the case of a public service corporation, at least, its plain duty to the public. If a different course were pursued the only method of pro-

* *Loc. cit.*, p. 10.

† *Loc. cit.*, p. 13.

viding for replacement of property which has ceased to be useful would be the investment of new capital and the issue of new bonds or stocks. This course would lead to a constantly increasing variance between present value and bond and stock capitalization—a tendency which would inevitably lead to disaster either to the stockholders or to the public, or both. If, however, a company fails to perform this plain duty and to exact sufficient returns to keep the investment unimpaired, whether this is the result of unwarranted dividends upon over issues of securities, or of omission to exact proper prices for the output, the fault is its own. When, therefore, a public regulation of its prices comes under question, the true value of the property then employed for the purpose of earning a return cannot be enhanced by a consideration of the errors of the management which have been committed in the past."

* * * * * * *

"After the company had closed its case the city undertook to determine the present value of the company's property by the plain method of ascertaining the cost of reproduction, diminished by depreciation. In its case in rebuttal, the company followed the same method, though the results differed largely, and, as we have seen, *no proper allowance for depreciation was made.*"

The United States Supreme Court, in Willcox *et al.*, constituting the Public Service Commission of New York, *vs.* Consolidated Gas Company, says:*

"And we concur with the court below in holding that the value of the property is to be determined as of the time when the inquiry is made regarding the rates. If the property, which legally enters into the consideration of the question of rates, has increased in value since it was acquired, the company is entitled to the benefit of such increase. This is, at any rate, the general rule. We do not say there may not possibly be an exception to it, where the property may have increased so enormously in value as to render a rate permitting a reasonable return upon such increased value unjust to the public. How such facts should be treated is not a question now before us, as this case does not present it. We refer to the matter only for the purpose of stating that the decision herein does not prevent an inquiry into the question when, if ever, it should be necessarily presented."

In the same case, the United States Supreme Court holds that a valuation of $12 000 000 for the franchise, to be added to a valuation of $47 000 000 for physical properties, is excessive.

This value was arrived at, by the lower court, by assuming a constancy of relation between the value of the franchise and the value of the

* United States Reports, Vol. 212, p. 52.

tangible property. The franchise value had been fixed in 1884, by agreement of the companies which consolidated, at $7 781 000. This valuation received some sanction or endorsement by a legislative committee, which investigated the consolidation in 1885, and expressed the opinion that this valuation of the franchise was not more than its fair value.

At the time of the consolidation the physical properties were valued at $30 000 000; the accepted value of the franchise at that time, therefore, was 26% of the value of the tangible properties.

By applying 26% to the increased valuation in 1906 of the tangible properties, or to $47 000 000, the lower court reached the conclusion that the franchise value had increased to more than $12 000 000, the value disapproved by the Supreme Court. The Court says:*

"But although the State ought, for these reasons, to be bound to recognize the value agreed upon in 1884 as part of the property upon which a reasonable return can be demanded, we do not think an increase in that valuation ought to be allowed upon the theory suggested by the court below. Because the amount of gas supplied has increased to the extent stated, and the other and tangible property of the corporations has increased so largely in value, is not, as it seems to us, any reason for attributing a like proportional increase in the value of the franchises.

"Real estate may have increased in value very largely, as also the personal property, without any necessary increase in the value of the franchises. Its past value was founded upon the opportunity of obtaining these enormous and excessive returns upon the property of the company, without legislative interference with the price for the supply of gas, but that immunity for the future was, of course, uncertain, and the moment it ceased and the legislature reduced the earnings to a reasonable sum the great value of the franchises would be at once unfavorably affected, but how much so it is not possible for us now to see. The value would most certainly not increase.

"What has been said herein regarding the value of the franchises in this case has been necessarily founded upon its own peculiar facts, and the decision thereon can form no precedent in regard to the valuation of franchises generally, where the facts are not similar to those in the case before us. We simply accept the sum named as the value under the circumstances stated."

The Supreme Court, in these recent opinions, does not refer to the method used in estimating the depreciation or amortization increment

* *Loc. cit.*, pp. 47 and 48.

which must have entered into the calculation of net return. If this was properly determined in the Knoxville case on the basis of the remaining useful life of the several parts of the water-works, then the opinion of the Court relating to the valuation in that case is eminently proper; but the statement of facts in connection with this point is not clear. Neither does the Court have anything to say about it. The Court no doubt assumed that the method of computation was a correct one. In other words, if error was committed at all it was not the error of the Court.

It may be assumed, therefore, that the decision was rendered just as it would have been if amortization had been correctly determined (as it may have been), and as far as the ultimate result is concerned, the decision of the Court is in accord with the principles which have herein been noted; but, for the sake of standardizing and simplifying the method of arriving at the desired result, the Supreme Court might with propriety, when opportunity arises, qualify the opinion expressed in the Knoxville case so that all questions of the permissibility, either to make the appraisal of value for rate-fixing purposes with depreciation deducted, or, as an alternative, to make the appraisal a fair appraisal of the amount of invested capital, using in each case the proper method of computing the amortization annuity, will be set at rest.

Comparison of Various Methods of Computing Interest and Amortization.

To make it clear that the two methods of valuation for rate-fixing purposes lead to identical results, a pipe line of mature age may again be used for illustration, and reference may also be had to the case of the steamboat already cited. The expectancy of the pipe line is 40 years; it has been constructed progressively one-fortieth each year. There will be one-fortieth of the pipe 40 years old. This has served its time and is of no value. Another fortieth has served 39 years, and its remaining value (after deducting the amount in the amortization fund due to this fortieth) will be 4.8% of the cost of replacing it. Another fortieth, 38 years old, will have a depreciated value of 9.5%, and so on. The last fortieth, being new, will have full value. The average value, as has already been stated, will be 63.8%, or $63.80 for each $100 of the total investment. The remaining life which must be assumed for an equivalent unit of this value is 18.0 years.

The computation, on the theory of valuation approved by the U. S. Supreme Court, will now be as follows:

Depreciated value of the pipe line for each $100 of the investment	$63.80	
Amortization increment to be applied annually, which will amount to $63.80 in the remaining 18.0 years at 4 per cent		$2.50
Net earnings on $63.80 at the assumed rate of 4% per annum		2.55
Total earnings in excess of operating expenses		$5.05

The computation, on the principle of valuing the investment without deduction for depreciation, will be as follows:

The investment will be	$100.00	
The amortization increment to be applied annually, which will amount to $100 in 40 years at 4% will be		$1.05
The net earnings on $100 of the investment at 4% will be		4.00
The total earnings in excess of operating expenses, etc.		$5.05

The earnings, including amortization, estimated by the two methods are identical. They are also identical in the case of a single depreciating item, as in the case of the steamboat at 10 years, or at any other period of its life. They will always be identical, whether the plant is large or small, simple or complex.

The simple method of making appraisals should, in the end, find general acceptance, and when the fact of the absolute agreement of this method with that laid down in the Knoxville case is properly brought to the attention of the Courts, it may be expected that it will obtain their approval.

In Table 8, and by the diagram, Fig. 3, the results of computing earnings according to five different methods are presented. All figures in Table 8 apply to $100 of invested capital. The interest rate on safe investments is taken at 4% per annum. Similar tables and diagrams could have been prepared for other expectancies than 20 years, and for other rates of interest, but this single table will suffice to make clear the fundamental principles which are involved, and particularly the fact that the results by the simplest method of all, No. 1, always coincide exactly with the results by Method No. 3, the latter being in unquestioned conformity with the opinion of the U. S. Supreme Court, as recently expressed in the Knoxville case.

The first method of computing earnings, as illustrated in Table 8 for a 20-year life at 4% per annum, as has been fully explained, is based on a valuation at all times at 100% of the investment. The amortization fund is supposed to be held as a part of the property, transferable with it, and the amortization increment is not written off as depreciation. The several amounts paid into the amortization fund are not available to the owner until replacement is necessary at the end of the term of the plant's usefulness. The moment they are applied as a retirement of capital, it becomes necessary to compute amortization for the remaining value and the remaining life. When this is done annually, the result is as shown under Method No. 3.

The second method is an approximation which is not generally applicable. It is proper for a complex plant of mature age, when it can be shown that there has been no opportunity to accumulate an amortization fund; when, in other words, the allowance for amortization has not exceeded the requirement for replacement. A modification of this method results from the application of the formula for replacement, as elsewhere noted, to be used in the case of plants of a uniform rate of growth.

The introduction of the annual replacement requirement properly determined by any method, in place of amortization, would make Method No. 2 of computing earnings generally applicable in all cases in which past amortization increments have not exceeded the replacement requirements.

The third method is that which literally conforms to the recent Supreme Court decisions, already quoted. Depreciation is taken into account in making the valuation, and amortization is estimated for this valuation and the remaining life. This agrees absolutely with Method No. 1. It is based strictly on the assumption that the amortization is synonymous with depreciation, and is deducted from the investment as each annual increment is received. The annual, gradually increasing amortization or depreciation increment, under Method No. 3, can be ascertained by formula as follows:

Let a represent the amortization annuity for the full or original expectancy (the same as under Method No. 1).

i represent the rate of interest used in determining the annuity.

n be the number of the year for the end of which the depreciation increment is to be estimated.

A_i represent the depreciation increment for the year, n:

Then: $$A_i = \frac{100\,a}{i}\left[\left(\frac{100+i}{100}\right)^{n+1} - \left(\frac{100+i}{100}\right)^{n}\right]$$

For $i = 4$, that is, for an interest rate of 4%, this will be:

$$A_4 = 25\,a\,(1.04^{n+1} - 1.04^{n})$$

For $i = 3$, there will be:

$$A_3 = 33.33\,a\,(1.03^{n+1} - 1.03^{n})$$

For $i = 5$, there will be:

$$A_5 = 20\,a\,(1.05^{n+1} - 1.05^{n})$$

TABLE 8.—METHODS OF CALCULATING ANNUAL INTEREST AND AMORTIZATION FOR AN EXPECTANCY OF TWENTY YEARS.

Method No. 1.

For each $100 of original investment. Interest 4 per cent.

At the end of year.	VALUATION=INVESTMENT WITHOUT DEDUCTION FOR DEPRECIATION.			
	Amortization Based on Expectancy.			
	Valuation for each $100 of investment.	Interest at 4% per annum.	Annual amortization increment.	Net earnings, including amortization.
0	$100.00	$4.00	$3.35	$7.36
1	100.00	4.00	3.36	7.36
2	100.00	4.00	3.36	7.36
3	100.00	4.00	3.36	7.36
4	100.00	4.00	3.36	7.36
5	100.00	4.00	3.36	7.36
6	100.00	4.00	3.36	7.36
7	100.00	4.00	3.36	7.36
8	100.00	4.00	3.36	7.36
9	100.00	4.00	3.36	7.36
10	100.00	4.00	3.36	7.36
11	100.00	4.00	3.36	7.36
12	100.00	4.00	3.36	7.36
13	100.00	4.00	3.36	7.36
14	100.00	4.00	3.36	7.36
15	100.00	4.00	3.36	7.36
16	100.00	4.00	3.36	7.36
17	100.00	4.00	3.36	7.36
18	100.00	4.00	3.36	7.36
19	100.00	4.00	3.36	7.36
20	100.00	4.00	3.36	7.36
Averages	$100.00	$4.00	$3.36	$7.36

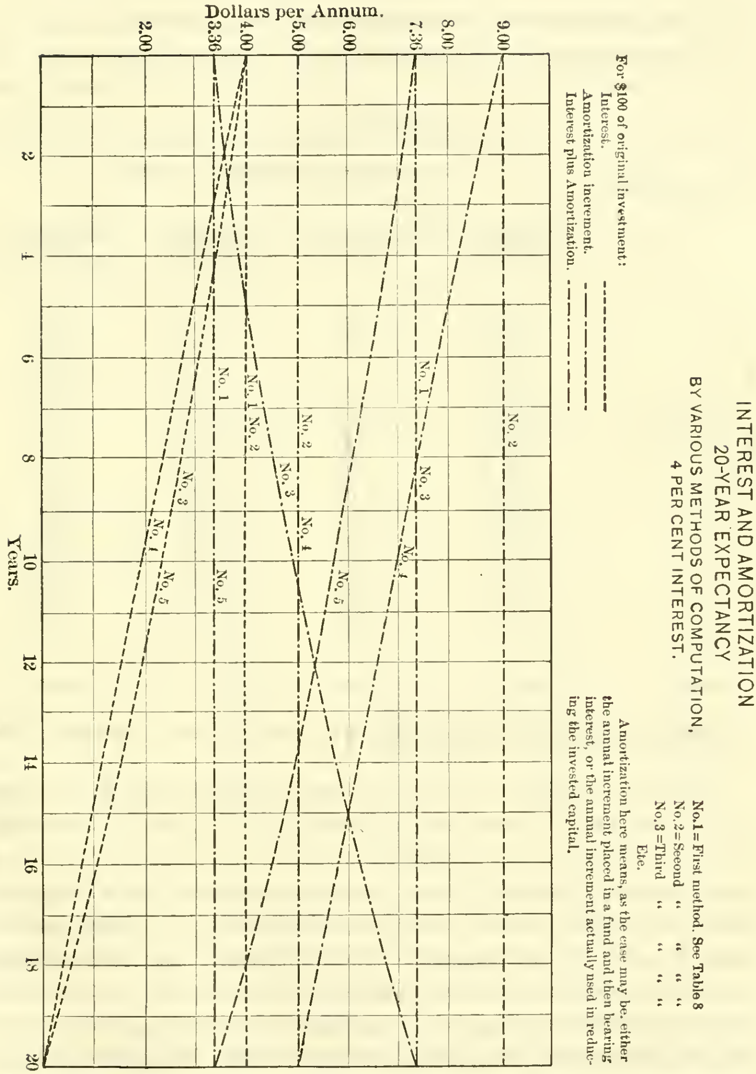
INTEREST AND AMORTIZATION
20-YEAR EXPECTANCY
BY VARIOUS METHODS OF COMPUTATION,
4 PER CENT INTEREST.
No.1 = First method. See Table 8
No.2 = Second " " " "
No.3 = Third " " " "
Etc.
Amortization here means, as the case may be, either the annual increment placed in a fund and then bearing interest, or the annual increment actually used in reducing the invested capital.
For $100 of original investment:
Interest.
Amortization increment.
Interest plus Amortization.
Dollars per Annum.
9.00
8.00
7.36
6.00
5.00
4.00
3.36
2.00
No. 1
No. 2
No. 3
No. 4
No. 5
2
4
6
8
10
12
14
16
18
20
Years.

Fig. 3.

TABLE 8 (*Continued*).—Methods of Calculating Annual Interest and Amortization for an Expectancy of Twenty Years.

Method No. 2.

(An approximation method applicable in special cases only.)

For each $100 of original investment. Interest 4 per cent.

At the end of year.	Valuation=Investment without Deduction for Depreciation.			
	Amortization by Straight Percentage.			
	Valuation for each $100 of investment.	Interest at 4% per annum.	Annual amortization increment.*	Net earnings, including amortization.
0	$100.00	$4.00	$5.00	$9.00
1	100.00	4.00	5.00	9.00
2	100.00	4.00	5.00	9.00
3	100.00	4.00	5.00	9.00
4	100.00	4.00	5.00	9.00
5	100.00	4.00	5.00	9.00
6	100.00	4.00	5.00	9.00
7	100.00	4.00	5.00	9.00
8	100.00	4.00	5.00	9.00
9	100.00	4.00	5.00	9.00
10	100.00	4.00	5.00	9.00
11	100.00	4.00	5.00	9.00
12	100.00	4.00	5.00	9.00
13	100.00	4.00	5.00	9.00
14	100.00	4.00	5.00	9.00
15	100.00	4.00	5.00	9.00
16	100.00	4.00	5.00	9.00
17	100.00	4.00	5.00	9.00
18	100.00	4.00	5.00	9.00
19	100.00	4.00	5.00	9.00
20	100.00	4.00	5.00	9.00
Averages	$100.00	$4.00	$5.00	$9.00

* In the case of a plant of mature age, made up of numerous parts, amortization as here noted is the annual replacement requirement.

The fourth method, based on a valuation which takes depreciation into account and allows an annual amortization increment determined by the direct percentage method, has the serious defect of requiring large earnings in the early years of a plant's life and smaller earnings toward the end of its life. This defect is fatal to its general application. It may find occasional application, however, as a convenient method of approximating the required earnings in the case of complex plants of mature age, but, even then, as is shown by the line of averages in the table, the probability is that it will be in some measure unfair to the owner. It will give results in strict accord with those of Methods Nos. 1 and 3 at only a single period of the life of a plant, as for example, at 4.2 years for a plant (or an item) having a 10-year life; at 8.2 years for a plant having a 20-year life, and at 14.5 years for a plant having a 40-year life, and so on.

The fifth method is incorrect, and is decidedly unfair to the owner. It is to be condemned under all circumstances, notwithstanding the fact that it gives nearly correct results in the early years of a plant's life.

TABLE 8 (*Continued*).—Methods of Calculating Annual Interest and Amortization for an Expectancy of Twenty Years.

Method No. 3.

For each $100 of original investment. Interest 4 per cent.

At the end of year.	Valuation=Investment Less Depreciation. Amortization Based on the Remaining Life.			
	Valuation for each $100 of investment.	Interest at 4% per annum.	Annual amortization increment.	Net earnings, including amortization.
0	$100.00	$4.00	$3.36	$7.36
1	96.64	3.86	3.49	7.36
2	93.15	3.73	3.63	7.36
3	89.52	3.58	3.76	7.36
4	85.94	3.43	3.93	7.36
5	80.81	3.27	4.09	7.36
6	77.73	3.11	4.25	7.36
7	73.48	2.94	4.42	7.36
8	69.06	2.76	4.60	7.36
9	64.46	2.58	4.78	7.36
10	59.68	2.39	4.97	7.36
11	54.71	2.19	5.17	7.36
12	49.54	1.98	5.38	7.36
13	44.16	1.78	5.59	7.36
14	38.57	1.54	5.82	7.36
15	32.76	1.31	6.05	7.36
16	26.71	1.07	6.29	7.36
17	20.42	0.82	6.54	7.36
18	13.88	0.56	6.80	7.36
19	7.08	0.28	7.08	7.36
20	0.00	0.00	7.36	7.36
Averages	$58.95	$2.36	$5.00	$7.36

The averages at the bottom of Table 8 are the valuations and amounts which apply in the case of equal groups of items of every possible age. The average amortization noted for Method No. 3 is the same as the amount which would be estimated by the straight percentage method. This is true for any life, not alone for the 20-year period, to which the table applies. In other words, when amortization has been properly allowed from the beginning, in the case of a complex plant as described, the earnings are to include interest on the remaining investment and amortization computed by the straight-percentage method.

In the case of a plant made up of many parts of various periods of useful life, the practice is sometimes followed of estimating depreciation for each group of parts of equal life (n years) at one-nth of the remaining book value.

Under such practice, the average book value of each \$100 of original investment, if the plant has mature age and its parts are uniformly distributed to all possible ages, will be about as follows:

In a group having a 5-year life	\$67.23
" " " 10 " "	65.15
" " " 20 " "	64.15
" " " 30 " "	63.83
" " " 40 " "	63.68

Interest and amortization (in this case the assumed depreciation) would be figured as follows:

5-Year Life:

Interest on \$67.23 at 4 %	= \$2.69	
Amortization, 20% of \$67.23	= 13.45	
	——	\$16.14
Whereas interest plus amortization should be, at least....		22.64

10-Year Life:

Interest on \$65.15 at 4%	= \$2.61	
Amortization, 10% of \$65.15	= 6.52	
	——	\$9.13
Whereas interest plus amortization should be, at least....		12.33

20-Year Life:

Interest on \$64.15 at 4%	= \$2.57	
Amortization, 5% on \$64.15	= 3.21	
	——	\$5.78
Whereas interest plus amortization should be, at least....		7.36

40-Year Life:

Interest on \$63.68 at 4%	= \$2.55	
Amortization, 2.5% on \$63.68	= 1.59	
	——	\$4.14
Whereas interest plus amortization should be, at least....		5.05

TABLE 8 (*Continued*).—Methods of Calculating Annual Interest and Amortization for an Expectancy of Twenty Years.

Method No. 4.

For each $100 of original investment. Interest 4 per cent.

At the end of year.	Valuation=Investment Less Depreciation.			
	Amortization Based on Straight Percentage.			
	Valuation for each $100 of investment.	Interest at 4% per annum.	Annual amortization increment.	Net earnings, including amortization.
0	$100.00	$4.00	$5.00	$9.00
1	95.00	3.80	5.00	8.80
2	90.00	3.60	5.00	8.60
3	85.00	3.40	5.00	8.40
4	80.00	3.20	5.00	8.20
5	75.00	3.00	5.00	8.00
6	70.00	2.80	5.00	7.80
7	65.00	2.60	5.00	7.60
8	60.00	2.40	5.00	7.40
9	55.00	2.20	5.00	7.20
10	50.00	2.00	5.00	7.00
11	45.00	1.80	5.00	6.80
12	40.00	1.60	5.00	6.60
13	35.00	1.40	5.00	6.40
14	30.00	1.20	5.00	6.20
15	25.00	1.00	5.00	6.00
16	20.00	0.80	5.00	5.80
17	15.00	0.60	5.00	5.60
18	10.00	0.40	5.00	5.40
19	5.00	0.20	5.00	5.20
20	0.00	0.00	5.00	5.00
Averages	$50.00	$2.00	$5.00	$7.00

The amortization increment computed by Method No. 5 is clearly inadequate.

It is now possible to prepare tables for various expectancies which will show the required earnings (not including any allowance for management), including amortization, computed by methods which have been shown to be proper.

Table 9 is based on Method No. 1 (Table 8). The property is appraised for rate-fixing purposes at 100% of the investment, and the original expectancy is made the basis of computing the annual amortization increment. In this table the interest column is not, strictly speaking, based on the true value of the property, neither is the amortization annuity noted in the following column in strict conformity with the growth of an annuity fund, but the sum of the two columns is the correct sum of these two increments which are to be covered by the earnings. (The same rate of interest is supposed to apply throughout.)

TABLE 8 (*Continued*).—METHODS OF CALCULATING ANNUAL INTEREST AND AMORTIZATION FOR AN EXPECTANCY OF TWENTY YEARS.

Method No. 5.

(Always erroneous.)

For each $100 of original investment. Interest 4 per cent.

At the end of year.	VALUATION = INVESTMENT LESS DEPRECIATION.			
	Amortization Based on the Full Expectancy.			
	Valuation for each $100 of investment.	Interest at 4% per annum.	Annual amortization increment.	Net earnings, including amortization.
0	$100.00	$4.00	$3.36	$7.36
1	96.64	3.86	3.36	7.22
2	93.15	3.73	3.36	7.09
3	89.52	3.58	3.36	6.94
4	85.74	3.43	3.36	6.79
5	81.81	3.27	3.36	6.51
6	77.73	3.11	3.36	6.47
7	73.48	2.94	3.36	6.30
8	69.06	2.76	3.36	6.12
9	64.46	2.58	3.36	5.94
10	59.68	2.39	3.36	5.75
11	54.71	2.19	3.36	5.55
12	49.54	1.98	3.36	5.34
13	44.16	1.78	3.36	5.14
14	38.57	1.54	3.36	4.90
15	32.76	1.31	3.36	4.67
16	26.71	1.07	3.36	4.43
17	20.42	0.82	3.36	4.18
18	13.88	0.56	3.36	3.92
19	7.08	0.28	3.36	3.64
20	0.00	0.00	3.36	3.36
Averages	$58.99	$2.36	$3.36	$5.72

Table 10 and Fig. 4 are based on Method No. 3 (Table 8). The property is appraised with deduction of depreciation. The appraisal thus made will, in the case of a property of mature age and a sufficiently large number of parts, conform with the figures in the "Remaining value" column. The annual amortization increment is computed by the use of amortization tables, from the remaining life and the remaining value. The results presented in this table are obtained by methods of valuation in accord with the recent decision of the U. S. Supreme Court; and there is perfect agreement in the ultimate result with those obtained by Method No. 1.

Table 11 is for use when a plant has attained mature age and no part of the invested capital has been repaid. The annual amortization increment is here equal to the annual replacement requirement. It is not, therefore, to be applied as a reduction of the investment. If, however, there has been a partial repayment of capital invested, as in the case of aid extended by bond issues or otherwise, then the appraisal should be correspondingly reduced.

All these tables are based on 4% per annum, as the rate of return on ordinary safe investments. The earnings, as noted in the tables, do not include any allowance for management, nor for unusual risk and the like, which are to be made in each case as circumstances may warrant, either as has been explained, by the subterfuge of adding arbitrarily assumed intangible values to the appraisal, or by making an addition direct to the interest rate which is applied to the appraisal.

TABLE 9.—INTEREST AND AMORTIZATION FOR ANY PLANT OF ANY AGE.

Method No. 1 (Table 8).

Generally Applicable.

For each $100 of original investment. Interest 4 per cent.

Expectancy, in years.	Appraisal.	Interest.	Amortization annuity.	Interest plus amortization.
5	$100	$4.00	$18.463	$22.46
6	100	4.00	15.079	19.08
7	100	4.00	12.661	16.66
8	100	4.00	10.853	14.85
9	100	4.00	9.449	13.45
10	100	4.00	8.329	12.33
15	100	4.00	4.994	8.99
20	100	4.00	3.358	7.36
25	100	4.00	2.401	6.40
30	100	4.00	1.783	5.78
35	100	4.00	1.358	5.36
40	100	4.00	1.052	5.05
45	100	4.00	0.826	4.83
50	100	4.00	0.655	4.66

It is worthy of note, in the case of numerous parts of the same expectancy uniformly distributed to all possible ages, as will be seen by reference to Table 10, that, when depreciation is estimated by the annuity method, and is properly deducted from the invested capital, under Method No. 3, the annual amortization or depreciation increment is the same as though determined by the straight-line method.

The method of appraisal and computation of earnings illustrated in Table 11 is substantially correct for a plant of mature age when the annual replacement requirement may be substituted for the amortization. It is not likely that there will have been an excess of income during the early years of a plant's service. The early years are generally lean years, which are ordinarily expected to produce less than the desired income. Therefore, apart from exceptional cases, it may be generally assumed that a plant when it has reached mature age should be earning the replacement requirement in addition to a reasonable rate of interest on the investment.

TABLE 10.—INTEREST AND AMORTIZATION.

Average values for plants of numerous parts uniformly distributed to all possible ages.

Method No. 3 (Table 8).

For each $100 of original investment. Interest 4 per cent.

Expectancy.	Remaining life of equivalent single item, in years.	Average remaining value.	Interest on remaining value.	Amortization annuity for remaining life.	Interest plus amortization.
5	3.0	$61.58	$2.463	$20.000	$22.46
6	3.5	60.30	2.412	16.667	19.08
7	4.0	59.38	2.375	14.286	16.66
8	4.5	58.83	2.353	12.500	14.85
9	4.9	58.45	2.338	11.111	13.45
10	5.3	58.23	2.329	10.000	12.33
15	7.6	58.18	2.327	6.667	8.99
20	9.9	58.95	2.358	5.000	7.36
25	12.0	60.03	2.401	4.000	6.40
30	14.1	61.25	2.450	3.333	5.78
35	16.0	62.53	2.501	2.857	5.36
40	17.9	63.80	2.552	2.500	5.05
45	19.7	65.10	2.604	2.222	4.83
50	21.5	66.38	2.655	2.000	4.66

If, however, it can be shown that the investment has been cut down by excessive earnings or by a direct repayment of capital, as in the case of municipal or State aid by contribution of funds to the owner, then the interest rate should be applied only to the remaining investment.

Method No. 4, Tables 8 and 11, is practically equivalent to a computation of the replacement requirement for inclusion in the earnings. When, however, a new plant of numerous parts is in question, all the expectancies of which are n years, it would be better to grade the replacement increment from nothing at the beginning to one-nth of the investment in the nth year.

TABLE 11.—Interest and Amortization.

For Plants of Mature Age in Case that the Amortization Earned in the Past has not Exceeded the Replacement Requirements.

Method No. 4 (Table 8).

For each $100 of original investment. Interest 4 per cent.

Expectancy.	Appraisal.	Interest.	Annual amortization or replacement requirement.	Interest plus amortization.
5....................	$100	$4.00	$20.00	$24.00
6....................	100	4.00	16.67	20.67
7....................	100	4.00	14.29	18.29
8....................	100	4.00	12.50	16.50
9....................	100	4.00	11.11	15.11
10....................	100	4.00	10.00	14.00
15....................	100	4.00	6.67	10.67
20....................	100	4.00	5.00	9.00
25....................	100	4.00	4.00	8.00
30....................	100	4.00	3.33	7.33
35....................	100	4.00	2.86	6.86
40....................	100	4.00	2.50	6.50
45....................	100	4.00	2.22	6.22
50....................	100	4.00	2.00	6.00

The practice has heretofore been so general of assuming that the amortization annuity, based on the original expectancy, was an adequate amortization allowance, that there may be cases in which such allowance was from year to year erroneously deducted from the investment as depreciation, Method No. 5, Table 8. Let the case be considered where this has been done for a plant with a 20-year useful life. When this plant is 10 years of age the owner will have received ten annual amortization payments of $3.36. These, under this assumption, were applied to reduce the capital invested when he received them. At the 10-year period, therefore, there is a value of $67.40 left in the plant for each $100 of original investment. The earnings should be interest on this amount at 4%, or $2.70 plus the amortization for $67.40 for the remaining 10 years, or ($67.40 $\times$ 0.0833) $=$ $5.61, making $8.31. If the plan of allowing interest on the reduced valuation plus the original amortization increment of $3.36 had been followed, the earnings of ($2.70 $+$ $3.36) $=$ $6.06 would be inadequate.

Conclusion.

Thus far, no distinction has been made between expectancy and the actual life. All computations have been made as though there were absolute conformity between the actual life and the expectancy.

This, however, is never strictly true, because some items of every group will go out of use before they are of mature age, while others will survive their expectancy.

The amortization annuity estimated for the actual life of a large number of items will not necessarily agree with the amortization annuity based on average probable life. That there must be disagreement will readily be seen when a single item is taken into consideration. This may be one of those doomed to fail early; or it may be one of the large number which will reach a mature age; or it may be among the smaller number which serves long after the expected age has been passed. Taking all probabilities into account, when the item is new, it will be found that the amortization rate which should apply will always conform to an actual life somewhat less than the expectancy.

It is not proposed to follow this matter further, nor to attempt a specific illustration which would necessarily have to be based on some assumption relating to mortality unsubstantiated by experience; but there may be found in this fact some justification for making liberal allowance for amortization from the beginning of a plant's service.

According to the Court opinions previously quoted, there is a distinct recognition by the United States Supreme Court of the propriety of including intangible values in the appraisal for rate-regulation purposes. The Court, however, indicates no method by which the value of a franchise is to be determined. It states distinctly that the opinion in the New York gas rate case is not to be considered a general precedent.

There is also a distinct recognition of the fundamental principle that the value of the investment should be maintained as at the beginning. This is strictly correct if it is intended to apply to the value of the properties as a business and not to the tangible properties alone. There is hardly room to doubt that this is the actual meaning intended to be conveyed, because a little farther on the Court says that the tangible properties of water-works and the like begin to depreciate on the day they go into use. Such depreciation of the tangible properties, as referred to by the Court, cannot be offset or made good by any amount of repair work, because, to all intents and purposes, the depreciated items may continue for a long time to be rendering just as adequate service, and often even better service, than when first installed. To be kept at 100% of the investment, the appraisal would at all times have to be the value of the physical properties plus the amortization fund.

The difference between the value of the tangible property and the invested capital might, perhaps, according to this interpretation of the language used by the Court, be a proper measure of the intangible values; but, if thus measured, they serve merely as an excuse for bringing the valuation up to the investment, and, in that event, the amortization increment must be based on the full expectancy of the plant. In applying such a principle, account must be taken of the gradually decreasing value of the intangible elements. The steamboat in the last year of its life would be valued at 7%, and the intangible values appurtenant to the steamboat business would aggregate 93 per cent. No one would pay more than 7% for the boat, yet, as has been demonstrated, the rates may properly be based on a valuation of the steamboat business at 100 per cent.

This, of course, is an extreme case, but it illustrates the principle. Perhaps no Court has ever been asked to allow so large a proportion of intangible value. Yet the principle remains the same, whether at 5 years the intangible value is 18% of the entire appraisal, or whether at 19 years it is 93% thereof. This undesirable feature should condemn the use of intangible values to bring the appraisal up to the actual investment.

It will be much better to use the equivalent and uniformly applicable method of determining by the best available means what amount of capital is properly and reasonably invested. The attempt to draw sharply the line which separates the tangible from the intangible value should be discouraged.

If it were customary to maintain the amortization fund, perhaps by investment in outside securities, as an integral and inseparable part of the property, growing as depreciation increases and subject to transfer with the property as a part thereof, in case of a sale, then the fundamental principle, already fully explained, that the appraisal at all times should be at 100% of the investment, would be readily understood. It would then be clear that the earnings of the amortization fund would go into the property for replacement purposes, and that at all times the owner would be entitled, in addition to such earnings, to a proper rate of return on 100% of the investment, that is to say, on the value remaining in the physical property plus the amortization fund.

The Supreme Court distinctly lays down the principle that, as a general rule, increase in value should go to the owner of a property. This is a confirmation of the views previously set forth. First, that, strictly speaking, the increase should be treated as reinvested earnings; second, that, under the difficulty which will always exist of predicting from past experience what the future may bring, it will rarely be possible to estimate the future increment of earnings due to appreciation with sufficient certainty to take it into account in estimating the prospective surplus or revenue over expense, and, whenever this cannot be done, such increase of value will, in fact, go, as the Supreme Court says it should, to the owner; but, when, as an exception to the rule, which exception is pointed out by the Court, the property has increased enormously in value, then the fairness of taking account of the increase as a part of the earnings becomes apparent.

The foregoing is based throughout on the assumption that the ordinary rate of interest on safe investments is alone taken into account, and that any addition to this rate will be made as a direct addition to the earnings computed at this ordinary rate. The addition may be expressed either in percentage of the original investment, or in percentage of the remaining investment, or true value, as ordinarily understood and as defined by the U. S. Supreme Court in the Knoxville case. The latter may be found desirable, when, as should ordinarily be the case, the amortization has been earned from the beginning, but, for this purpose alone, a close estimate of actual value is not essential.

The results presented in Table 8 are made the basis of the curves shown in Fig. 3. Particular attention is asked to the lines marked No. 1 and No. 3. The sum of the ordinates of the two No. 3 lines, representing "interest" and "amortization," is always the same, and agrees throughout with the sum of the ordinates of the horizontal "interest" and "amortization" lines for No. 1. The diagram indicates plainly the extent of the departure of the other methods of calculation from the correct ones.

Method No. 1, according to which the appraisal for rate-fixing purposes is the investment without any reduction for depreciation, has certain advantages over Methods Nos. 3 and 4, the only other

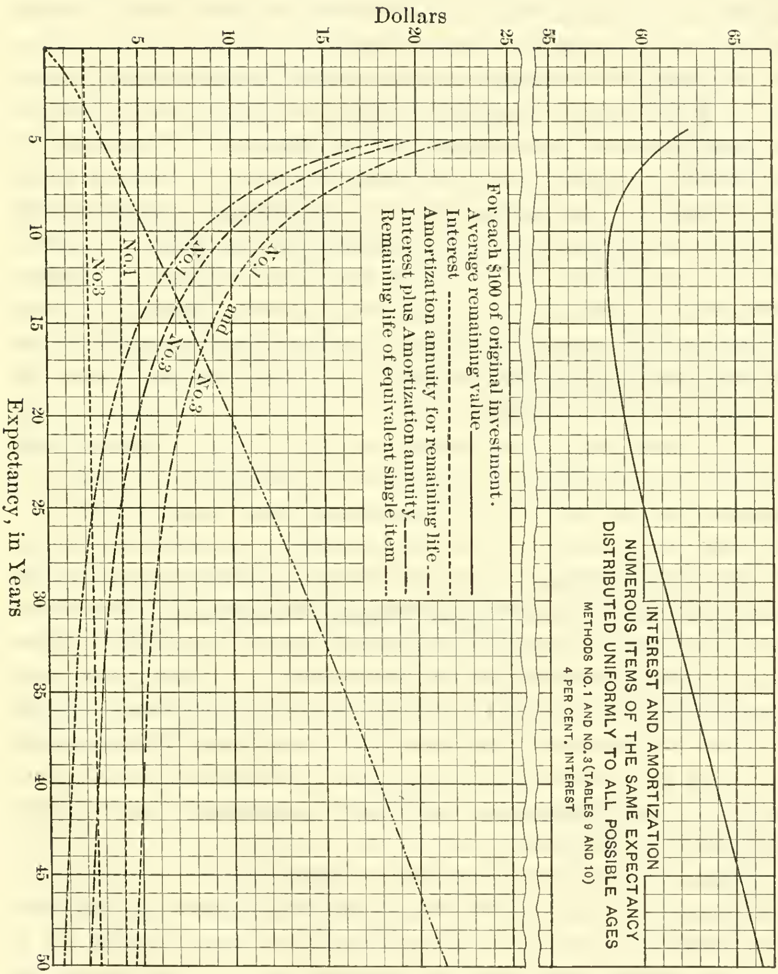
Dollars
Expectancy, in Years
For each $100 of original investment.
Average remaining value
Interest
Amortization annuity for remaining life
Interest plus Amortization annuity
Remaining life of equivalent single item
INTEREST AND AMORTIZATION
NUMEROUS ITEMS OF THE SAME EXPECTANCY
DISTRIBUTED UNIFORMLY TO ALL POSSIBLE AGES
METHODS NO. 1 AND NO. 3 (TABLES 9 AND 10)
4 PER CENT. INTEREST
No. 1
No. 3
No. 1 and No. 3

Fig. 4.

strictly correct methods of calculating allowable earnings, which may be briefly stated as follows:

Method No. 1 is always applicable when it can be shown that earnings have been adequate in the past, no matter whether the property is a single item, or is composed of many items; whether the expectancy is long or short; whether the expectancy is uniform for all parts of the property or not; whether the plant is of mature age or not; whether the property has attained full growth or whether it is still growing at a uniform rate, or otherwise. It is simple of application, and does not involve determination of the present condition of the property, provided that it is maintained in proper condition to render adequate service. It furnishes all the information necessary for intelligent action in fixing rates, because when it is known what the net earnings, above operating expenses, must be to yield the return which money should earn in ordinary safe investments, then an arbitrary addition can be made, to compensate the owner for management and risk of loss.

In contrast with these advantages, Method No. 3, under which "value" as ordinarily understood must be determined by deducting depreciation from the investment, requires a special determination of value for each item of which the property is composed, and a new determination every year for every item, or in special cases, for every group of items of the same expectancy. Each item has a new value each year and a remaining life which grows continually shorter. Amortization, therefore, must be estimated on a new basis each year. The judgment of the expert is called into play to determine the condition and probable remaining life of the several parts of the property, and after the complex calculation is made, if the same basic rate of interest is used throughout, the result should agree absolutely with the simpler Method No. 1.

When, for any reason, the rate of interest to be earned on the investment is higher than the rate of interest applied to the amortization annuity (in estimating depreciation) then, under Method No. 3, the net earnings will follow a descending scale. The rates to be charged, full compensation being assumed, will be higher in the early years of a plant's life than in its last years. This is an undesirable feature, resulting from the application of Method No. 3. It is avoided under Method No. 1. Herein is found an additional reason for the general adoption of the method of appraisal for rate-fixing purposes under which no deduction from the invested capital need be made for depreciation.

An advantage that may properly be claimed for Method No. 3 lies in this, that it discloses, more or less approximately, the part of the capital remaining in the property, and therefore, actual value of its tangible elements as such value would be estimated by a purchaser.

Method No. 4 not only has the same disadvantages as No. 3, but it is not acceptable, as has been explained, owing to the decrease of the earnings with increasing age of the plant. Its results do not, as do those of No. 3, agree from year to year with those of Method No. 1.

No. 5 is a method of approximation which is applicable whenever a plant, made up of numerous parts, has mature age, and it can be shown that in the past the earnings have been inadequate to supply an amortization fund, in excess of replacement requirement, which fund approximates 40% of the total investment in perishable properties; and when no part of the invested capital has been otherwise returned to the owner.

When the practice shall have been established of writing off nothing from the investment for depreciation, there will be a modification of the ordinary system of keeping accounts. It will then be desirable to open an "amortization and replacement account," which will be debited with the amount of the amortization actually earned, and there will be credited against it every item of replacement. The discarded items will be credited to the account of "invested capital" at their cost, and this account will be debited with the cost of the new items which replace the old.

It will be practical, too, to combine the replacement and the repair accounts whenever for any reason this may appear desirable. The depreciated value, according to the book accounts, under such a system, can at any time be found by subtracting the amount in the amortization fund from the invested capital.

DISCUSSION

Mr. Bamford.

WILLIAM BROKAW BAMFORD, M. AM. SOC. C. E. (by letter).—It is axiomatic, for the correct application of the principle set forth by the author, or for the equitable appraisal or valuation of any property, that the "expectancy" or probable life of the property be determined within reasonable limits. The writer is contemplating the presentation of a paper on the probable life or endurance of property, and, for the purpose of this discussion, will point out only certain general principles.

The "expectancy" or "actual life" of any property or individual can only be determined by summarizing the probable "endurance" of the various elements which may affect in any way the "actual life." For a property, as for an individual, the result at best is but an approximation. Nevertheless, it is possible to tabulate facts obtained over a series of years so that tables of the probable life of a property can be prepared with as reasonable a degree of accuracy as those which determine the probable life of the individual.

Until methods for properly forecasting the actual life of property are put on as stable a basis as the preparation of mortality tables for human life, we will have difficulty in adjusting equitably and scientifically, the various financial questions connected with public service as well as private property.

For the purpose of clearness in discussing the subject, the writer has preferred to devise and use the term "endurance" rather than "actual life," "expectancy," or "depreciation."

The "endurance" of a property may be said to be its power and ability to prolong its life or existence against the adverse forces or influences of any kind which threaten it. It is its power to remain in the same state without perishing. The endurance may be considered as being ultimately established by the "actual life" of the property.

It is indisputable that if a property has no endurance it cannot last, and that its endurance is due to various factors which tend to prolong or shorten its life. The elements which influence the endurance of a property might be divided into the physical causes which threaten its existence as a structure (depreciation or deterioration) and the various economic or commercial causes which threaten its life as a property (obsolescence). In addition, there are certain hazards which threaten its life, which might be summarized as follows:

(*A*). Physical Endurance of Property (depreciation or deterioration)—threatened by:

(*1*) stability of structure;

Mr. Bamford.

(*2*) physical deterioration;
 (*a*) in materials composing structure,
 (*b*) in methods of construction.

(*B*). Economic or Commercial Endurance of Property (obsolescence)—threatened by:
 (*1*) obsolescence due to improvements in machinery, processes, etc.;
 (*2*) obsolescence due to lack of serviceability for use;
 (*3*) obsolescence due to changed industrial and social conditions;
 (*4*) actual cost too excessive for present value.

(*C*). Hazards to Endurance of Property:
 (*1*) destruction by fire;
 (*2*) destruction by act of God—earthquake, cyclone, flood, etc.;
 (*3*) destruction by domestic violence or foreign wars.

Thus it will be seen that "endurance" or actual life is threatened and controlled by elements composing the three divisions of (*A*) physical endurance, (*B*) economic endurance, and (*C*) hazards. The actual life, therefore, will be terminated by the element which has the least amount of "endurance." In a rapidly growing community and progressive age, economic endurance is usually shorter than physical endurance. In the present age evidences of this are seen on every hand, obsolescence terminating the actual life of property the physical endurance of which may still be of "indeterminable duration."

All questions of depreciation, as usually considered at the present time, should be resolved into attempts to establish a standard for the endurance or actual life of the property. In the majority of engineering investigations of the endurance of property, however, the primary efforts are directed toward solving the endurance of the physical or structural elements, to the neglect or subordination of the economic or commercial elements and hazards.

Neither the physical nor the economic endurance can be considered alone in determining the actual life of property; both must be determined, together with the hazards, and that one having the least endurance will be the one to govern the case in question. The probable physical life is no guide to the probable economic life; while the actual economic life is a positive check to the actual—not the potential—physical life.

It is true, of course, that the probable economic life is more difficult to determine accurately in advance than the probable physical life; but, as with all forms of insurance, it is possible to apply the law of averages so that the resultant will be equitable to the public service companies as well as to the public.

Mr. Bamford.

In its practical application it would be advisable to have a State commission to establish uniform standards for the determination of probable economic life and "endurance," which should be subject to periodic adjustments. Such a procedure is not novel, as the English Local Government Board has undertaken just such a work in establishing periods for the redemption of authorized bonds for local improvements.*

Without the possibility of establishing the "expectancy" or "endurance" of property equitably, the very admirable methods proposed by the author will prove valueless. It is hoped that in the future more attention will be given to "endurance," rather than to concentrating efforts solely on physical "depreciation."

Mr. Oxtoby.

James V. Oxtoby, Esq. (by letter).—The writer has read this paper with interest, and wishes to express his appreciation of the work which Mr. Grunsky has done toward making clearer the facts regarding the element of depreciation in making appraisals of public utility properties.

It is conceded that an owner of a public utility is entitled to earn a reasonable return on his investment. He must also receive from the business, on its being wound up, or on its sale, an amount equal to the principal of his investment. If depreciation of plant is provided for on a sinking-fund basis, the interest which the depreciation fund earns is part and parcel of that fund, which must be added to it annually in order to bring it up to the required amount at the end of the depreciation period. Otherwise, the fund becomes at once impaired. The annual amount paid into the sinking fund by the business is smaller than it otherwise would be, because it is expected that the earnings of the fund will be added to it, and that, by compounding the interest yearly, the fund, at a predetermined date, will equal the amount of the depreciation. If the owner uses any part of this fund, whether principal or interest, he is really using part of the principal of his investment.

In appraising a plant for rate-making purposes, its value is its reproduction value, and not its reproduction value less depreciation. The entire depreciation must be earned from the public. If the sinking-fund method is used in computing the amount of the yearly depreciation, it is apparent that the fund must exist and must be made to earn its own increment. Some students of this question have stated that the depreciation fund has already been earned from the public, and that including the depreciation fund in the valuation would be requiring the public to pay dividends on moneys already paid in by it This is not true where the sinking-fund method is used. By that method the public is asked to pay, not the full amount of the yearly

* *Engineering News*, Vol. 54, p. 462.

depreciation, but only a part thereof, the compound interest earned on what the public has so paid making up the deficiency. Mr. Oxtoby.

This confusion of ideas seems to result from the failure to distinguish between the repayment and withdrawal of investment from year to year, and the establishment of a sinking fund which does not mature until some date in the future. In the first case the investor has got back his money and can apply it to other uses. In the second case his money is withheld from him, being locked up in the sinking fund; and he is not only unable to apply it to other uses, but must see to its being invested in absolutely safe securities not affected by depreciation, in order that at maturity the fund may be adequate to meet the liability for which it was provided.

A similar confusion of ideas affects many students when they are called on to state a fair price for a utility expropriated as a going concern. They err in stating the price as reproduction less depreciation, the vendor to retain the depreciation fund. In practice the depreciation fund is locked in securities which may or may not be then marketable for their assumed value. The law authorizing expropriation should impose on the purchaser the burden of marketing these securities promptly, or of purchasing them himself. Otherwise, the investment made by the vendor is not fully released. In this the writer intentionally omits reference to the equitable requirement of a higher price in expropriation proceedings than in rate-making proceedings.

Assume an investment of $25 000 made in a plant, which, at the end of twenty years, will have a depreciated or junk value of $5 000. Assume 6% as fair return on the plant investment and, for convenient figuring, assume that it is possible to invest a sinking fund in safe securities paying 6 per cent. Table 12 is based on these assumptions, and on the further assumption that the sinking-fund method of calculating depreciation is correct.

From the foregoing it is evident that if the owner is required to accept as profit less than $1 500 yearly, he has been deprived of a reasonable return on his $25 000 investment.

A plant probably could not continue to operate usefully without replacement before the end of the depreciation period, but this illustration, taken with the many which Mr. Grunsky has furnished, demonstrates the principle that if the annual depreciation is computed on a sinking-fund basis, it is inequitable to use depreciated value alone as the basis for the reasonable return to which the owner is entitled.

If the depreciation is computed on the straight-line basis, the interest which the depreciation fund earns is really an earning. If depreciation thus computed is paid to and withdrawn by the owner, he is entitled to earn his fair return only on the remaining value.

Mr. Oxtoby.

The owner of a public utility, however, is bound to keep up the plant to a high degree of efficiency and must rehabilitate it when necessary, which practically precludes withdrawal.

TABLE 12.

Year.	Value of plant at end of year, by assumption.	Return of 6% on $25 000.	Annual depreciation sinking fund, 6% basis.	Interest on fund at 6%.	Total depreciation fund at end of year.
0	$25 000.00	....	....	....	....
1	24 456.31	$1 500.00	$543.69	....	$543.69
2	23 880.00	1 500.00	543.69	$32.68	1 120.00
3	23 269.11	1 500.00	543.69	67.20	1 730.89
4	22 621.58	1 500.00	543.69	103.85	2 378.42
5	21 935.22	1 500.00	543.69	142.70	3 064.78
6	21 207.60	1 500.00	543.69	183.88	3 792.40
7	20 436.37	1 500.00	543.69	227.54	4 563.63
8	19 618.82	1 500.00	543.69	273.81	5 381.12
9	18 752.46	1 500.00	543.69	322.86	6 247.54
10	17 834.16	1 500.00	543.69	374.85	7 165.84
11	16 860.52	1 500.00	543.69	429.95	8 139.48
12	15 828.46	1 500.00	543.69	488.37	9 171.54
13	14 734.48	1 500.00	543.69	550.29	10 265.52
14	13 574.86	1 500.00	543.69	615.93	11 425.14
15	12 345.07	1 500.00	543.69	685.50	12 654.93
16	11 042.09	1 500.00	543.69	759.29	13 957.91
17	9 660.93	1 500.00	543.69	837.47	15 339.07
18	8 196.90	1 500.00	543.69	920.34	16 803.10
19	6 645.02	1 500.00	543.69	1 008.19	18 354.98
20	5 000.00	1 500.00	543.69	1 101.33	20 000.00

Mr. Grunsky demonstrates that if the depreciation fund is used for replacements, which themselves at once commence to depreciate, the sinking-fund method is impracticable. The fact is that depreciation is not an exact quantity, but must be determined by the exercise of fair judgment. Its amount at any time is the difference between actual investment and present value. It is highly improbable that, during a series of years, the actual depreciation of a plant will coincide with the accumulation of money by a sinking-fund rule. A public service plant should be maintained from its earnings so that its present value will always equal its original cost. If not, the difference should be in a depreciation fund, by whatever method it is computed; and a fair return should be computed on the total value which serves the public, whether this value is in plant or in depreciation reserve. Real earnings by the depreciation reserve are a part of the earnings of the plant. The increment, however, of a fund which is accumulating on a sinking-fund basis is not an earning.

In this discussion the writer has, in general, merely stated in his own way ideas which appear in the paper. His appreciation of the argument contained therein has been his excuse for doing so.

Mr. Higgins.

Charles H. Higgins, M. Am. Soc. C. E.—This paper should receive the careful attention of the members of this Society, for it deals with matters far more fundamental than a reading of the title or introduction might lead one to believe. In fact, nowhere has the speaker found the main issue stated clearly, but a student of these matters cannot read this paper carefully without finding the author constantly returning to this undefined issue. Whether it be veiled purposely or merely clouded because not clearly seen by the author, it cannot escape being the point of such overwhelming importance in this paper as to challenge attention. Mr. Higgins.

On page 773 occurs the following sentence:

"Perhaps the use of the term, 'value,' in this connection is unfortunate, because it is not clear why 'value,' as ordinarily defined (which is not always synonymous with capital reasonably and properly invested), should be made the criterion of allowable earnings."

And, again, on page 820:

"* * *, and to the ruling of the Courts, which hold that owners of public service properties are entitled to a fair return on the 'value' of such properties.

"If it be found that the ruling of the Courts is not subject to modification, or, in other words, that appraisals must be 'value,' as 'value' would be determined by a purchaser, that is to say, for the tangible elements in most cases, cost or cost of replacement less depreciation, or something practically equivalent thereto, * * *."

In these lines the real meat of the matter may be found, the author taking issue with the decisions of the United States Supreme Court.

Stripped of all verbiage, this paper deals then, not with the methods of making appraisals of public service properties under the existing law as interpreted by the Supreme Court, but with what, in Mr. Grunsky's opinion, the law should be. The latter may well be an equally proper matter for discussion before this Society, but it is certainly very different from the former.

To present this matter clearly, the speaker will illustrate. As early as 1898, in the leading case of Smyth *v.* Ames,* in the Nebraska maximum rates cases, the Supreme Court laid down the principle that the basis of all calculations, as to the reasonableness of rates, must be the fair value of the property used, and specified certain matters to be taken into consideration in ascertaining the fair value: the original cost of construction, the amount expended in permanent improvements, the amount of market value of the bonds and stock, the present, as compared with the original, cost of construction, the probable earning capacity of the property under the particular rates prescribed, and the sum required to meet operating expenses; all to be given such weight as would be just and right in each case. Justice Harlan was careful to add: "We do not say that there may not be other matters to be regarded in estimating the value of the property."

* 169 U. S., 466.

Mr. Higgins.

The following year, in the case of the San Diego Land Company *v.* National City,* the Court held "what the Company is entitled to demand in order that it may have just compensation is a fair return upon the reasonable value of the property at the time it is being used for the public." The Supreme Court, then, as early as 1899, had adopted present value as the standard, leaving undetermined how a reasonable value is to be ascertained and what constituted a fair return.

Again, in 1903, in San Diego Land and Town Company *v.* Jasper,† the Court said:

"It no longer is open to dispute under the Constitution that what the Company is entitled to demand, in order that it may have just compensation, is a fair return upon the reasonable value of the property at the time it is being used for the public."

In a masterly review of the subject of regulation of railway rates, Judge Swayze, of the Supreme Court of New Jersey, says:

"Novel questions of this character will arise with increasing frequency, and require the most careful consideration. Like most other questions in every department of law, they are in their origin rather questions of fact than questions of law, although in course of time the rules become settled and thus become rules of law. In their origin and as yet many are questions of sound business management and engineering science. The law prescribes reasonable return upon a reasonable valuation. What is a reasonable return and what is a reasonable valuation must vary with the circumstances of each particular case."‡

It may be accepted then as an established rule that the appraisal of a public service property to be used in fixing rates should show the fair value of the property.

Now, Mr. Grunsky argues that for the fair "value" of the Supreme Court there should be substituted something which he calls "capital properly and reasonably invested," stating, as a fundamental principle, that:

"The valuation of a public service property and its earnings must bear such relation to each other that there will be returned to the owner, within the life of the property, the capital which he has properly invested in it, and in addition thereto, interest at a reasonable rate, upon such amount of capital as from time to time actually and properly remains in the property as an investment."

In this Mr. Grunsky takes issue with the Fourteenth Amendment as interpreted by the Supreme Court since 1898.

The author's "fundamental principle" quoted above, if applied, as

* 174 U. S., 739.

† 189 U. S., 439.

‡ *Quarterly Journal of Economics*, May, 1912.

outlined in the paper, would cut both ways. In determining what is termed "capital reasonably and properly invested" according to the author's plan, earnings and dividends, or interest, are considered apparently from the beginning, and, if these dividends have been less than what is determined as a reasonable rate of return, the difference is considered as remaining in the property as invested capital. On the other hand, in the case of past earnings above the determined rate, Mr. Grunsky speaks as follows: "It is possible, of course, in the case of large earnings in the past, that a portion thereof should be considered as capital returned to the owner." In other words, the author would appear to propose, not only the determining by a State of a reasonable rate for the present and future, but actually to make it retroactive. Mr. Higgins.

Mr. Grunsky seems inclined to treat all investors as if they were owners of State bonds. For a lucid explanation of the relative positions of the holders of different classes of securities and their relation to the public, nothing better in its way can be found than the Report of the Railroad Securities Commission, of which President Hadley, of Yale University, was Chairman.

To the speaker's mind, the Courts, in fixing on fair valuation as the basis for all calculations, have taken the only position economically sound.

The speaker accepts the conclusion of Judge Swayze already quoted: "The law prescribes reasonable return upon a reasonable valuation. What is a reasonable return and what is a reasonable valuation must vary with the circumstances of each particular case." This states clearly the proposition before an engineer making an appraisal or valuation for use in the fixing of rates under the law as it exists to-day, and there are enough technical difficulties yet to be settled by engineers. The broader question dealt with in this paper is very interesting, but should not be confused with methods under the law as established.

Henry Floy, M. Am. Soc. C. E.—The speaker cannot agree with the views expressed by Mr. Higgins. Mr. Grunsky has brought out quite clearly a very important question, which must still be fought out and decided by the Supreme Court, and that is, the basis of "fair value." The various opinions rendered by the Supreme Court have not yet fairly and squarely determined the question as to whether or not "fair value" shall be taken as that derived from a consideration of accruing, theoretical depreciation, or something in addition to such value. The speaker is inclined to agree with the author that the investor is entitled to a return on his investment, or the cost of reproduction, if his property is kept in good operating condition. Present value accurately Mr. Floy.

Mr. Floy. obtained from a consideration of accruing depreciation must vary from day to day, thus forming a fluctuating and impractical value on which to base rates or capitalization. Such fluctuating value is too unstable and unfair ultimately to be received and accepted for fair value, as the author has brought out.

It is very difficult, even for lawyers, to interpret the decisions of the Court, and surely it is much more difficult for engineers to ascertain exactly what these decisions of the Supreme Court which have been referred to—the Consolidated Gas case and the Knoxville Water case—mean in the last analysis. It would seem as if, by these decisions, the Court intended to convey the idea that something more than a theoretically depreciated value should be made the basis on which a return is to be allowed.

It is possible that, if "the recall" is established, some new decisions may be expected; yet, at the present time, we may feel confident that the Courts, regardless of how excessive the returns have been in the past, would maintain the position that present and future returns will be based on the fair value of the property. The Courts will not attempt to reduce earnings in the present below the fair return on the fair value of the property, even though the owner for some years previously has been earning more than a fair return.

A return of 4% has been mentioned in the discussion, but the Supreme Court has never yet named anything less than 6%, and that for conditions such as exist for a monopoly in New York City. Of course, a fair return for any particular property depends on that property, and one like the Consolidated Gas Company in New York City—a monopoly in the greatest city in America—is not running as much risk in the way of securing a return as some other property in a small town in the West, and the Courts and Commissions have recognized this fact. The Commission of New York City, for example, has in certain decisions explicitly stated that certain gas and electric corporations were entitled to a return of 7½ or 8% on the total value of their properties. This means that the stockholder may obtain a very much higher rate of interest for his holdings, while the bondholders, who have a prior lien on the property, are willing to accept a 4, 5, or 6% return for their share of the investment.

In the matter of valuing real estate, the Courts have quite generally held that, in determining present value, the owner is entitled to appreciation, and this theory was distinctly enunciated in the Consolidated Gas case, where the U. S. Circuit Court approvingly quotes the language of the Supreme Court in another case to the effect that "the value of the property at the time it is being used" should be taken.

It must be recognized that fair values may be different for different purposes. The property of a corporation which is to be assessed for the purpose of special franchise tax in New York City, for example, would have a different value from that computed for other purposes, because a franchise tax relates only to the property in the street, which may be a relatively small part of the total property of the corporation. The value of property for rate-making purposes, in a similar way, may not be the same as that for capitalization, because a part of the property may be held simply as an investment or in connection with an allied business, so that the value for rate-fixing purposes, in such case, would be quite different from the value proper for capitalization. These questions of "fair value" are of comparatively recent origin, and are by no means easy of solution. Until a few years ago, no one appreciated or considered the fact that the operation of public utility property was a matter of much public concern. Public utilities were given grants and encouraged to make investments with the expectation of large returns, certainly 8 or 10%, possibly 15 or 20%; but lately, as the corporations have developed and become in many instances monopolies, either through crushing out competition or buying up their competitors, the public has been compelled to deal with the situation on a basis radically different from that formerly allowed. This has resulted in the development of what may be called a theory, which we are attempting to make a science, with regard to the control and operation of public utility properties, so that the situation is quite different from anything that existed fifteen or twenty years ago. To-day we look upon a public utility corporation as entitled to special privileges, such as the use of public streets, the right of condemnation, permission to exist perhaps as a monopoly, in return for which, in view of some assurance of reasonable profit, we limit the return on the value of the property to a fair amount, which, while greater than that received from a Government bond or municipal security, is nevertheless smaller than would be judged reasonable for an industrial concern, where the risks are greater. In brief, the peculiar conditions surrounding any specific corporation must be considered both in fixing the fair value of the property and fair return thereon. Mr. Floy.

J. Martin Schreiber, M. Am. Soc. C. E.—The author has presented a remarkable analysis, and has approached the subject in a liberal manner, which is certainly required for a rate investigation, if a sound conclusion is to be expected. Mr. Schreiber.

The greatest difficulty is in the practical application of the principles which have been set forth. In discussing the paper by Mr. Riggs,* the speaker brought up the question of fair values in relation to present physical values. Also, in a recent investigation of the depreciation of certain elements of physical property, he interviewed

* *Transactions*, Am. Soc. C. E., Vol. LXXII, p. 1.

Mr. Schreiber. leading specialists eminently fitted to give the information desired. Some of these men replied that they would not even attempt to designate the correct life, if changes in the art were to be considered; and, of course, an estimate of the life of property without the consideration of obsolescence would be valueless. To take a practical case: It is reported* that, in the recent valuation of the Elevated Railroads of Chicago, the value, exclusive of roadway and overhead charges, submitted by the Chicago Harbor and Subway Commission, was $26 354 217; the appraisement by the A. L. Dunn Company was $40 750 892, and the valuation by George F. Swain, M. Am. Soc. C. E., was $34 634 396. Now, it does not appear to be fair to the stockholders to be asked to abide by any method showing such variations. It should be admitted that a fair valuation, representing moneys properly invested for property which has been built up by the piecemeal method, with equipment constantly changing on account of the development of the art, along with unreliable book records, with their varying accounting methods, is a very difficult proposition. This is especially true when one must take into consideration the vague item of overhead charges; thus far, that item has generally been determined by setting apart an arbitrary percentage. For this reason the speaker is of the opinion that a valuation, particularly of old properties, for rate purposes based on earning power, is more reliable than the method which takes the values of actual physical properties as the governing data. Also, the value for rate making should not, necessarily, be ascertained by the same plan as the value for taxes or bonds.

Assuming that a fair valuation is known, another very pertinent practical question is: "What is to be allowed to the stockholder?" The speaker believes that the general tendency is to place too low a limit on proper earnings. One should not expect to raise money to finance projects which carry with them a large element of risk at savings account dividends, even if the utility appears to be reasonably safe. The investor knows full well that there are still such contingencies as strikes, earthquakes, floods, and lean years, and that it is absolutely impossible to anticipate these conditions; besides, if one expects the country to develop, one must also encourage the honest man with the brains and energy.

Mr. Boucher. William J. Boucher, Assoc. M. Am. Soc. C. E.—During a part of 1911, the speaker had certain work in connection with an appraisal of the lines of the Illinois Central Railroad in Illinois. The values given to the right of way in Chicago, from Randolph to 12th Streets, presented some interesting phases.

When the road was constructed, about 1853, this section was built on piles and over the waters of Lake Michigan, to a terminal at Randolph Street. This space has since been filled in; the width of the

*Electric Railway Journal, May 18th, 1912, p. 829.

right of way varies from 100 to 400 ft., is depressed some 12 ft. below the general level, and is contained between retaining walls, beyond which, on the east and west sides, lies Grant Park. The railroad occupies a most unique and valuable grant for about 7 miles south from Randolph Street, lying, as it does, along the shore of the Lake, giving space for eight running tracks, a roundhouse and car-cleaning and storage yards. It is a popular notion, which the speaker cannot verify, that the land cost the railroad company nothing, but was granted as an inducement to enter the city. Mr. Boucher.

The westerly edge of the right of way through Grant Park is 500 ft. or more from the property line of Michigan Avenue, the nearest property which is ever for sale and has a definite, taxable value. The Company, in its appraisal, placed a square-foot value (seemingly very high) on all the right of way, in addition to improvement values, such as filling, which the Company had done, beginning at Randolph Street and increasing toward the south until it reached a maximum opposite Jackson Boulevard, where property values along Michigan Avenue are the highest, and then decreasing again until 12th Street was reached.

The question then arises: How were the values arrived at for property which had probably never cost the railroad company anything, which is located 500 ft. from the nearest property for a comparison of values, and which to-day could not be acquired for railroad purposes in any manner or under any circumstances? And, further, is a railroad company justified in placing a value in excess of $36 000 000 on such a property for appraisal purposes?

Another matter regarding railroad finances has often interested the speaker: One frequently sees, in the daily press, announcements of the sale of bonds of various railroads, which read somewhat as follows:

"The A. B. & C. R. R. is offering for sale $1 000 000 of 4% bonds, secured by various underlying securities (mentioned). Of this amount, $500 000 is to be applied to renewals of bridges and rails and $500 000 for new cars and new locomotives."

The question that arises is this: Is such use of a bond sale proper, if depreciation on rails, bridges, cars, and locomotives has been charged off? Why not use the funds from such depreciation account? Why is it proper or necessary to issue bonds for these ordinary renewals, which in the usual course of events must be taken care of, and must be expected? It has always seemed to the speaker to be improper. Where is it going to lead the corporation, and where will it end, to keep adding to the capital account and paying interest on worn-out and scrapped material?

R. D. Coombs, M. Am. Soc. C. E.—In a hypothetical case, such as a corporation having paid for a number of years a very high dividend Mr. Coombs.

Mr. Coombs. rate, should those dividends be deducted from the capital before fixing the amount on which they could properly be paid?

To consider an exaggerated case, suppose some corporation had been paying about 50% dividends for four or five years; if the Court decides that the corporation can charge only such a rate as will enable it to pay a fair percentage on present value, must those 50% dividends for four or five years be deducted?

Mr. Van Cleve. A. H. Van Cleve, M. Am. Soc. C. E.—Mr. Grunsky has undoubtedly rendered a great service to the Engineering Profession in setting forth so clearly the methods to be used in the appraisal of public service property for rate-making purposes. While there are marked differences of opinion as to the methods which should be used in such cases, the author's examples and tabulations illustrate so completely the principles which he advocates that his results and the methods of reasoning which lead up to them are entirely clear. His statement of fundamental principles is especially valuable, but the speaker begs to differ with him in respect to Principle No. 18 as applied in a water-power development the output of which is used for supplying electrical current to the public. That principle is stated as follows:

"Intangible values should be disregarded, in making appraisals for rate-fixing purposes, excepting only when the rate of net return is deliberately fixed at or too near the rate earned by ordinary safe investments, in which case an arbitrary addition to the appraisal, under whatever name, should be made. The interest on this item of the appraisal will be the reward of the owner for management and for any hazard which the business may involve."

In the case of a water-power development, the value of the water right, which is an intangible value, is one of the most important items to be considered in determining the value of the plant, not only for purposes of sale, but for the determination of the rates which may be properly charged by the owners of that plant for the service which they render; and the consideration of this item is essential, regardless of the question of the rate earned by ordinary safe investments. To neglect the value of a water right would work a grave injustice to its owner, for, in many cases, if only the tangible values of the several parts of the plant are considered by the rate-making body, and the returns to the owner are estimated at an ordinary rate of interest thereon, together with due allowance for amortization and all other items which may properly be included, it would result in compelling him to furnish service to the public at a rate far below that for which the same service could be furnished by a plant producing the same power from any source other than water.

It is true that the author, in Principle No. 17, states: "Proper investments for franchises, for water rights, and the like, are always to be included in the appraisal." It is frequently the case, however, that, although no investment has been made directly for the acquisition

Mr. Van Cleve.

of the water right, the value, nevertheless, exists, and should always be considered in a valuation. The correctness of this principle may perhaps be most clearly demonstrated by an example:

Let it be assumed that A is the owner in fee simple of lands lying between two streams, the water in one of which is at a material elevation above that in the other; that this property has been in his possession for many years, and that the price paid for the land was the fair and reasonable value of it for farm purposes; that A as a riparian owner has the right to withdraw water from the upper stream and discharge it into the lower stream; and that he has complied fully with all the legal requirements necessary for this purpose. A is then the possessor of a water right for which no expenditure has been made by him. Let it be assumed, further, that in a city located within reasonable transmission distance there is a market for the electrical current which can be produced by the water which may be withdrawn from the upper stream adjoining A's property and discharged into the lower stream, and that A determines to grasp the opportunity to make use of a water right which, before the advances in the art of transmission, was practically valueless, owing to the lack of any market in the immediate vicinity of his property. Having obtained the necessary franchise in that city, in due course he enters into contract for the sale of this power to the municipality and to the citizens thereof, and the plant which he has built, therefore, becomes a public service property. In the course of time a rate-making body is called on to decide as to the fair and just returns which he shall receive on his investment, and the value of the water right becomes—as it will be shown—by far the most important factor in determining those rates. In order to simplify the illustration, let it be assumed that the just returns are to be determined for power at the city limits after the voltage has been reduced for safe distribution, and thus eliminate from consideration the cost of the distributing plant.

It is found that the plant is capable of developing at all times 25 000 e.h.p. at the switch-board, and that the original cost and also the replacement value of the tangible values included in the plant are equivalent to \$40 per e.h.p., that the corresponding tangible value of the transmission line is \$6 per e.h.p., and that of the step-up and step-down transformer station and switch-board apparatus represents an investment of \$14 per e.h.p., or a total of \$60 per e.h.p. of actual investment in tangible property. The load factor is found to be 50%, and 13% the average loss in transmission and transformation. The power available for sale at the city limits, therefore, is 21 750 e.h.p., and the average power sold throughout the year is 10 875 e.h.p., or 43.5% of the total installed capacity.

Assuming the average life of the plant to be 20 years, and that 5% is a fair and reasonable return on the investment, the annual fixed charges and operating expenses per electric horse-power would

Mr. Van Cleve. then be fixed by the body called on to appraise the property for rate-making purposes about as follows, the figures being based on the investment per electric horse-power:

Interest on \$60, at 6%	\$3.60
Amortization, 3.36%	2.02
Taxes and insurance, $1\frac{1}{2}$%	0.90
Maintenance and repairs	1.00
General expenses	0.70
Operation (wages and supplies)	1.00
Total	\$9.22

This figure is based on the total capacity of the plant, and is equivalent to \$21.20 per electric horse-power of the average power sold, or less than one-third of a cent per kilowatt-hour. In other words, if the owner of the property were allowed a fair and reasonable return on his actual investment in tangible property, he would be required to furnish power at the city limits at less than one-third of a cent per kilowatt-hour, a result which is manifestly absurd. It is evident that, although his actual investment in tangible property may have been determined properly, nevertheless, that investment does not in any sense represent the real value of his holdings, on which the rate of return should be determined, and that this error is due to the omission from consideration of the value of a water right which cost him nothing.

Furthermore, it would certainly seem to be a poor law which would omit from consideration, in an appraisal for rate-making purposes, a value on which the owner is taxed; and the Courts of New York State, in the case of The People *ex rel.* Niagara Falls H. P. & M. Co. *vs.* Smith,* have held that a riparian right is taxable. To deny an owner a return on a value for which he is taxed, whether that value is tangible or intangible, is a principle which certainly would not hold in law.

The correctness of including the value of a water right in the value of property on which a franchise tax is to be levied has been upheld in the case of The People of the State of New York *ex rel.* *vs.* the New York State Tax Commissioners.† This case was carried to the Court of Appeals, and the contention of the State Board of Tax Commissioners was fully upheld. It may be interesting to note that, in this case, not only was the value of the water right given due consideration as affecting the value of the entire plant of the power company, but it was further held that the value of that water right should properly be apportioned to the several parts of the plant in the same ratio as that which the tangible values of those parts held to the

* 70 App. Div., 543; 175 N. Y., 469.

† At an extraordinary term of the Appellate Division, November 22d, 1909, Hon. L. W. Marcus, Justice, presiding.

tangible value of the entire plant. The principle of including in the value of a water-power plant the value of the water right, therefore, has not only the approval of common sense, but the sanction of the Courts. Mr. Van Cleve.

Assuming that the value of a water-power is to be considered in the appraisal of a property for rate-making purposes, the question then arises as to the method by which that value shall be determined. If an actual investment has been made for the acquisition of the right, and it is held that the original cost represents a fair and reasonable value thereof, the case is a simple one, and the principle to be applied has been most clearly and ably set forth by Mr. Grunsky; but, on the other hand, if no outlay has been made for the water right, as in the illustration just given, the problem is a complex one, and no doubt there would be a great difference of opinion as to the manner in which the true value should be ascertained.

The speaker ventures to suggest a method which he has used, and believes determines fairly the procedure to be followed. Reverting to the foregoing illustration, first determine, for the sake of comparison, what annual returns a rate-making body would allow the owner of a modern steam plant for the development of the electrical current for which a sale could be effected in the city under consideration, namely, 10 875 e.h.p., average use. Let it be assumed that the investment per electric horse-power of rated capacity is $60; but, as the total capacity of the plant, for the purpose of comparison with the water-power development, need be only 21 750 e.h.p.—the amount of power which the latter could deliver at the city limits at a suitable voltage—the actual investment per electric horse-power for purposes of comparison should be $52.30. It is understood, of course, that, in determining the rate of return which the owner of a steam plant should receive on his investment, the higher value should be used, but, as the comparison is between a steam and a water-driven plant, the lower value will be used in this illustration. As the life of a steam-driven plant will be less than that of the water-driven plant, it will be assumed, for the sake of comparison, that the life of the former is 15 years.

The comparative fixed charges and operating expenses for the steam plant having a capacity equivalent to that of the water-driven plant will be approximately as follows:

Interest on $52.20, at 6%	$3.13
Amortization, 5%	2.61
Taxes and insurance, 1½%	0.78
Maintenance and repairs	1.30
General expenses	0.70
Fuel (coal, $2.25 per gross ton)	13.13
Wages and supplies	2.00
Total	$23.65

Mr. Van Cleve. The speaker recognizes the fact that there may be a material difference of opinion as to the amount which should properly be charged to the items of maintenance, fuel, and wages, but it is believed that the foregoing figures are conservative in that they do not exaggerate the operating expenses of a steam plant, under the conditions set forth, as compared with those of a water-driven plant. This is shown further by the fact that the total annual fixed charges and operating expenses per electric horse-power are equivalent to only 0.7 cent per kw-hr. of power development, which is certainly a reasonable figure for a plant of the capacity assumed.

Assuming the correctness of the foregoing illustrations, it would appear that the annual return which would be allowed by the rate-making body to the owner of a steam plant would be $23.65, while, if the value of the water right be disregarded, the owner of the water-driven plant would be allowed to receive for exactly the same service only $9.22, an annual difference of $14.43. The unfairness of such a proposition is self-evident. If the owner of the water-power plant is to be allowed to receive a like return for like services, the value of the water right must be determined by capitalizing the difference between the annual yearly expenditures as above set forth, which is $14.43. If it has been determined that the reasonable interest on money invested is 6%, a like rate should be used in capitalization. If, however, the value of the water right is to be taxed at 1%, the rate of capitalization to be used should be 7%, on which basis the value of the water right is $206.14 per e.h.p. of rated capacity, an amount far in excess of the investment per electric horse-power in tangible property. To omit this value from consideration in appraising the property for rate-making purposes would lead to a far greater error than to omit the entire value of the tangible property.

While the speaker is not aware that the Courts have passed definitely on the correctness of the foregoing principle for determining the value of a water right, this method has been brought before the Courts in the case of the Fulton Light, Heat and Power Company *vs.* The State of New York, and in the Franchise Tax case previously referred to. From the award given in the former case, it would appear that the Courts certainly gave grave weight to the above method which was used in determining the value of a water right. In the finding of the trial judge in the case of The People of the State of New York *ex rel.* *vs.* The New York State Tax Commissioners, and in the decision of the Court of Appeals thereon, no definite approval of the foregoing method for determining the value of the water right was given, but, on the other hand, there was no criticism of it, and it was concluded definitely that the right had a value which should be considered. Owing to the fact that the appellant set forth no theory as to the value of the water right, but stated merely that he had owned the canal for 30 years, and did not know what it was worth, it was shown that there was no necessity for the Courts to pass on the correctness of the theory advanced by the respondents. Mr. Van Cleve.

While the speaker is aware that opinions will differ as to the methods to be used in determining for rate-making purposes the value of water rights, he has suggested the foregoing in the hope that it may lead to discussion and suggestions, and the advancement of other theories.

There can be no doubt that the question is one which cannot be passed over lightly, and that it will become of increasing importance in the future, as water-power development is proceeding rapidly, and engineers will be called on more and more frequently, in making appraisals for rate-making purposes, to determine the value of the water right on which the owner of a public service corporation is entitled to returns. The fact that the water right cost the owner nothing, or that his investment for its acquisition was merely a nominal one, should have no real bearing on such determination.

The speaker is aware that the illustration which he has used herein may be considered as an extreme one—although supported by fact in actual plants now in operation—but the real question is not as to whether the details are in accord with the opinions of all, but rather as to whether the general principles to which attention has been called are correct.

W. Kiersted, M. Am. Soc. C. E. (by letter).—The painstaking manner in which Mr. Grunsky has treated the question of the appraisal of public service properties, particularly for rate-fixing purposes, is decidedly interesting at a time when the valuation of public utilities occupies so prominent a part of the work of engineers associated with municipal and public service corporations. The views appertaining to matters of valuation in its various aspects are certainly at variance, although perhaps not exactly divergent. Much is being written on the subject, and the efforts of one who has given appraisal as much study as the author appears to have done, in order to outline fundamental principles and proper rules of action, are certainly most welcome, and should receive the hearty encouragement of every one interested therein. Mr. Kiersted.

In 1897, the writer's paper, "The Valuation of Water-Works Property,"* and the discussion thereon, developed a divergency of views which was to be expected at that time. Since 1897 the many calls on engineers to value water-works properties and other public utilities, for rate-fixing, purchase, and other purposes, have so inspired the study of the various problems entering into the valuation of public utilities that to-day sufficient experience and knowledge should have been

* *Transactions*, Am. Soc. C. E., Vol. XXXVIII (1897), p. 115.

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accumulated to admit of outlining more uniform methods of valuation for any and all purposes than have prevailed heretofore, were the experience in this line of work to be directed methodically, through concerted action, to outlining essential and fundamental principles and general rules of work.

There may be no intentional bias on the part of most engineers, acting individually, in their efforts to formulate fundamental principles and rules affecting, and guiding in, the appraisal of public service properties for rate-fixing and other purposes; but is it not true that the individual sometimes becomes imbued almost involuntarily with the idea that certain methods or theories representing the fruits of much labor and of frequent use are right simply because they are familiar; is it not true that frequently the individual discovers himself attempting to adapt lines of practice to a theory worked out in a sort of academic way, instead of first studying thoroughly the practical side of the questions involved and formulating rules and methods in accordance with well-established and good lines of practice; and is it not true that the assumptions or illustrations often set forth to illustrate a theory are in part or wholly incompatible with the ordinary and permissible lines of practice, particularly in arguments presented in defense of a theory?

It seems to the writer that one of the essential things to be done, in advance of expending efforts to prescribe rules and methods to guide in the valuation of public utilities, is to determine what the fundamental principles governing questions of this kind may be, not necessarily those elaborated in the office, but those determined through an intimate knowledge of all the questions and all the problems entering into the organization, construction, operation, maintenance, and development of any public utility. Until these principles shall have been fully determined and clearly defined, it is difficult to conceive how any practical theory or rules of procedure can be laid down.

In almost every article or discussion relating to the valuation of public utilities, the term "life" is freely used, as though a public utility was constructed for the purpose of serving a specific want only for some definite period of time. This is fundamentally wrong, for every public utility must live as long as there is need for the use of it, whether the period of time be one year, a century, or longer. The error is confusing, and arises from applying to a composite structure a line of reasoning relating solely to the life limitations of various elements or units going to make up that structure, apparently without regard to its wholly indeterminate life. It is known from experience that certain units of a composite property subject to heavy wear and tear become useless in time, others become incapacitated and are no longer able to perform the functions which are expected of them as part of that property, and others are abandoned because progress in the arts

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and new inventions compel the substitution of new methods and new devices for those in use. In this manner, experience guides in placing a life limit on particular units of a composite property; but the same experience does not sanction the placing of similar limitation on the life of a public utility considered as a whole. The unit may have an approximately determinate life, while the composite structure possesses a wholly indeterminate life.

For instance: a municipality is a complex and composite organization, under State and Federal regulation, made up of individual units, each unit performing some particular function in the body politic; each unit contributes to the support and progress of the municipality; and each unit, while concentrating efforts toward its own business success, is contributing constantly to the progress of the organized unit in which it lives. All the units are blended into one great regulated force, under such regulations and restrictions for mutual support that the elimination, by death, failure, or disappearance, of any unit of the human fabric no more than temporarily affects the progress of the community as a whole. Every municipal public utility is organized and developed for the public need; it thrives upon and is in every way a part of the community. If the conditions which promote the prosperity of the community flag, it may cease to grow, and may even retrograde, creating a business depression which is felt by all the public service corporations, and affects in one way or another even the humblest individual. On the other hand, the prosperity of the one follows in the wake of the prosperity of the other. There is no limit of life to the community and its public utilities, as far as human vision goes, but there is a limit to the life and period of usefulness of the individual units in the community and of the personality of those engaged in operating its public utilities.

In like manner, the various elements going to make up a composite property like a water-works, street-car system, or similar public utility contributing to the welfare, advancement, comfort, and convenience of the community, stand in precisely the same relation to the composite structure as the human element does to the community in which it lives, prospers, and performs some specific function.

It seems to the writer, therefore, that a fundamental error is committed when the term "expectancy," or "life," is applied to any composite structure as it would be applied to an element or unit of that structure which is known from experience to have a more or less fixed and definite period of service. The structure as a whole is an enduring one, as long as there is a demand for it in the community which it serves. To remain serviceable, it must be extended and improved from time to time; its worn-out units must be replaced with new, larger, and better ones; new methods of operation must be substituted for old and antiquated ones; business organizations must improve, in order to meet

Mr. Kiersted. the demands of advancement in economics; and betterments must be made from time to time, to meet the various rules and regulations imposed by those who have been made the guardians of the public welfare and the public health and comfort. In short, any public utility, to be of service to the community which it is organized and constructed to serve, must be maintained in an efficient, up-to-date condition; and, in order to do this, renewals, replacements, and extensions must be made periodically. It cannot die; and it cannot be allowed to retrograde, as long as the community depending on it lives and prospers and the need for it continues.

Again, the progress of any community is by no means uniform. Certain conditions promote a rapid growth at one period which adverse conditions at a subsequent period may check temporarily. Such periods of growth and stagnation arise in the development of any public utility; nearly every decade registers a jump in structural costs arising from numerous replacements and general betterments in the line of progress. A composite property possessing an average age of 15 years may within 2 or 3 years have this average age nearly halved. Very seldom, in a progressive city, does the average age exceed 20 years. An illustration of how quickly the average age of a composite structure may change is that of a water-works property which was purchased in 1909 at an appraised value of $1 100 000, in round numbers, when the average age was 15.5 years. In 1912, after extensions had been made aggregating more than $820 000, the average age of the physical property was 8.7 years. The original construction of this property was started in 1883; extensive changes were made in 1886, again in 1898, and again in 1909. Between the periods of heavy expenditures there were longer periods of a more moderate rate of expansion.

Another case illustrating the irregular growth of a public utility is that of the water-works of Kansas City, Mo. The original works were built by the National Water-Works Company about 1875, with a water-supply connection in the Kaw River; in 1886 new water-supply works were constructed on the banks of the Missouri River, and, at the same time, extensive general improvements were made; in 1895 the city purchased the water-works from the private company, and made some large extensions; in 1905 further extensive and costly improvements were made, followed by other costly improvements in 1911. Aside from these expensive periodical extensions, pipe-line extensions progressed continuously, although somewhat irregularly. In 1912 all the pumping engines purchased in 1895 had been replaced by larger and more expensive machinery; one of the pumping stations had been entirely abandoned, and the other had been modified extensively. The purchase price in 1895 was $3 100 000; the cost of the physical property to-day is $8 000 000, in round numbers; and the average age is approximately 17 years.

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Both these illustrations are of public utilities in cities where the growth and development may have been more rapid than in some cities in other localities; but, however this may be, they afford a practical example of how public utilities are developed—the difference in the rate of development, between a rapidly growing and a slowly growing city, being one of degree only. Replacements and rehabilitation periods may be at more frequent intervals in a city of rapid growth than in one of slow development, and the average age of a composite property may be somewhat longer in the latter than in the former, but there can be no such condition in practice as a public utility standing still and progressively growing old. If a community retrogrades, the value of the individual units of city property, of the city property as a whole, and of all its public utilities, must depreciate; and the owner of a public utility must witness the value of his investment grow less and less as the community continues to retrograde.

These two considerations, namely, the indeterminate life of a composite structure like almost any public utility, and the comparatively short average age under almost any conditions to be found in any one of our American cities, are believed to be essential and fundamental, and to underlie all practical methods of appraising public utilities.

The estimated cost of duplicating the physical property of a public service corporation is not in itself a measure of the value of the property; it is simply an essential element of the value of such a property. The more completely replacements and substitutions of perishable and incapacitated units may have been made and the entire plant may have been maintained in a serviceable condition, the nearer does the fairly estimated cost of the reproduction of the physical property represent its full reproduction value as an element of total value of the entire property. It is seldom, however, that this is the situation when an appraisal is to be made, and consideration should be given to the measure or degree in which the physical property falls short of this ideal condition.

The author regards real estate, or rather the increment of value of real estate over and above the purchase price, as reinvested earnings open to consideration in an appraisal for rate-fixing purposes. This view seems to be entirely consistent with that generally entertained at the present day, and with the theory of valuation based on the cost of reproduction of the physical property under conditions as of the date of valuation. It would follow, naturally, from the same line of reasoning, that any of the physical conditions which increase the actual cost of reproducing any unit or any aggregation of units of the physical property, as compared with the cost at the time of construction, may also be considered with equal consistency as reinvested earnings, and, accordingly, should be included in the capital account subject to consideration for rate-fixing purposes. For instance: the

Mr. Kiersted. cost of laying water pipes under paved streets and in streets where the working force is subject to great interference on account of the presence of underground structures, notwithstanding the fact that the pipes were actually laid in unpaved streets before the existence of many of the present-day underground structures, has usually been regarded as a proper element of value in connection with cases where the valuation has been for purchase and sale, and has been occasionally questioned in connection with rate cases solely. It would seem, however, that if enhancement of value in one particular is to be considered in rate cases, enhancement of value in other directions should be open to equal consideration. Furthermore, if enhancement of value in any particular is legitimate and fully consistent with the theory of valuation on the basis of reproducing the physical property under present-day conditions, it would appear that, on the other hand, it is also perfectly proper that all questions of depreciation of the value of the physical property should be given equal weight. If it is fair to allow enhancement of value, it must be equally fair to consider depreciation of value; both are part and parcel of any method of valuation based on the cost of reproduction under the physical conditions existing as of the date of valuation.

The computation of depreciation, as applied to the appraisal of public utilities, has been rendered somewhat complex and rather inconsistent by the infusion of sinking-fund methods. The limited franchise under which many of the public utilities have been constructed and operated may have served to suggest the use of the sinking-fund method in computing depreciation. The application might be proper enough were the property to pass out of existence at the expiration of a limited franchise, and were the rates to be charged for public and private service sufficient to return to the investor the invested capital with interest; but it is seldom or never the case that the physical property expires with the franchise. There have been a very few instances where an investment has been irreparably injured by a city (after refusing either to purchase the property or renew an expired franchise) constructing competitive works and virtually crowding the owner of a public utility out of business. Occasions of this kind are so few as to be scarcely worthy of consideration in a comprehensive view of the subject, and have usually resulted from either an unfortunately drawn franchise ordinance or a bitter contention precipitated between the owner of the public utility and the city.

Sinking-fund computations find well-defined application to financial problems like those of the redemption of municipal, state, and national bonds, and to problems like those involved in life insurance; but the application of such a method to the financial management of any public utility compels a complication of accounts which scarcely any management of a public utility would care to introduce. Its applica-

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tion in this regard is not only cumbersome, but conflicts with the actual conditions of practice. Moreover, it is likely to prove inequitable. It compels the assumption of a life period for each of the numerous units entering into the composition of a property, and the computation of depreciation by well-known sinking-fund methods for some definite portion of an assumed life term for each of the various units.

The assumption of a life period for the units of a composite property and the computation of depreciation of the various units on the sinking-fund basis is equivalent to computations of depreciation on composite property having an equivalent life period. The operation of the sinking fund, as applied to the depreciation of a property having a composite life of 60 years, is shown by Table 13.

TABLE 13.

Age of property, in years.	Percentage of total depreciation, with sinking fund invested at 3 per cent.	Average annual rate of depreciation by decades.	
0	0		
10	7	0.7	1st Decade.
20	16.5	0.95	2d "
30	29.2	1.30	3d "
40	46.2	1.7	4th "
50	69.2	2.3	5th "
60	100.0	3.1	6th "

The average of the six decades is 1.67% per year, corresponding with the straight-life method of depreciation for a 60-year life.

The average rate of depreciation as thus computed is irregular; it is moderate in the earlier years and very rapid in the later years of the life of any unit, and, in the long run, is likely to result in an inequitably proportioned depreciation, a progressive increase of rates to meet interest on sinking-fund investments, and any number of incongruities in accounting and in attempting to harmonize practice with theory. The fact is that the sinking-fund method of computing depreciation assumes the plant to grow old continuously, and finally to wear out and pass out of existence, whereas, in reality, the plant usually grows more efficient as it grows older, through necessary and indispensable replacements of its various units, and the property becomes more valuable.

The sinking-fund method really has no place in computations of the rate of depreciation of a composite property. As long as worn out, incapacitated, and obsolete units and methods of operation are replaced progressively with new, improved, and larger units and methods, and the property as a whole is extended and enlarged from time to time, to meet the demands of a growing community, there seems to be little

Mr. Kiersted. ground for further consideration of depreciation, unless it may be that referred to as deferred maintenance.

By deferred maintenance is meant deterioration other than that which can be taken care of in the ordinary course of events by current expenditures covering ordinary wear and tear and ordinary replacements. For instance: as time goes on, the discharging capacity or serving capacity of the distributing pipes in a system of water-works is reduced materially by the tuberculation of their interior walls, and this necessitates a cleaning process, or reinforcements, or replacements. Such deterioration is usually allowed to progress until inferior service compels the use of comprehensive and extensive cleaning and replacement.

The average annual cost of replacements of four water-works properties, the histories of which are well known to the writer, when distributed over the average age period of the respective properties, is 0.6% of the cost of reproduction.

In another instance, a water-works property, which had been started in 1865 and in 1907 was found to have many of the older pipes heavily coated internally, received a special depreciation of 0.33% per annum of the cost of reproducing the physical property, representing deferred maintenance.

The aggregate annual cost, covering replacements and deferred maintenance, as above stated, of 0.93% of the cost of reproduction, represents a rate of depreciation which is under rather than over that which may be expected to approximate the rate of depreciation of these particular properties, for the reason that some of the minor replacements are not accounted for.

It may be stated, further, that the foregoing replacements are of the important parts of water-works, like pumping machinery, buildings, boilers, settling basins, water-supply intakes, and other important units which are subject to replacement or radical changes at long intervals. They do not in any degree embrace the expenditures covering ordinary repairs and current maintenance. These costs, wholly in excess of that above estimated, would by themselves approximate 0.8 to 1% per annum of the cost of reproducing the physical property. In all probability an allowance of 2% on the cost of reproducing the physical property of a water-works would represent the amount of money to be set aside annually for general maintenance; about half of this would be used to cover current maintenance expenditures, the other half could be set aside as a fund to replace the worn-out and incapacitated units, as becomes necessary in the ordinary course of events.

The writer does not presume to offer these figures for general application, although, for the particular properties considered, and for similar ones, they may not be far wrong. They serve the purpose of illustrating what the writer believes is a simple method of computing the depreciation of the physical part of a water-works property, and one which will commend itself to the bookkeeper as well as to the superintendent and other officers operating a public utility of this particular kind. By varying the annual percentage rate in harmony with the average rate of decay of the various elements going to make up the physical property of public utilities of other types, the same method may become equally applicable. Part of the money annually set aside from the earnings for general maintenance may be apportioned to meet the annual costs of current maintenance and repairs; another part may be a fund for use in making replacements, and for any similar work which would serve to perpetuate and insure the highest degree of efficiency and the continued usefulness of the physical property. Mr. Kiersted.

A straight-line method of this kind for computing depreciation avoids all the incongruities and inconsistencies of the sinking-fund method; it is equitable; it is practical, and in accord with the present methods of operating public utilities; and it simplifies bookkeeping, and can perhaps meet general requirements in this direction more nearly than any other.

In estimating the present value of the physical property of any public utility, it would simply be necessary to estimate the average age of the composite property which is being appraised, and deduct, as depreciation from the estimated cost of reproduction, an amount found by applying the annual percentage rate covering replacements and deferred maintenance multiplied by the age of the composite structure. The annual percentage rate should be determined from general experience and from particular knowledge of the property under appraisement.

The monopolistic character of many public utilities eliminates from consideration the question of the influence of competition to such a degree as to render the method of computing depreciation herein proposed more nearly applicable than would be the case were a property subject to the unrestrained influence of sharp competition, as may be the case in many private lines of business.

Passing to the question of the value of a public utility: It is clear that a distinction may exist as to the value for rate-fixing purposes and for purchase and sale, particularly when circumstances are such that franchise value can properly be taken into consideration. Where the power to regulate rates is exercised in accordance with law by a municipality annually, as in California, or by a commission, there can be no franchise value; but it does not follow that such an element of value of a public utility as that ordinarily termed "going value" or "going concern value" can be similarly excluded, for the simple reason

Mr. Kiersted. that it should not properly be classed as part of the intangible value, particularly when computed irrespective of earnings. Where income—or, more properly speaking, net income—is used as a basis and means of computing "going value," so called, it is open to this criticism, to a degree at least, because it may partake in part of the nature of franchise value, if the prevailing rates are high and the assumed period of time covered in the use of earnings in computing going value is abnormally long. In no event should an appraisal for rate-fixing purposes consider any element of value based on earnings or income. It does not follow, however, that there is no element of value over and above the cost of reproducing the physical property together with usual overhead charges which it is customary to add to such computations, which should be as open to consideration in appraisals for rate-fixing purposes as for purchase and sale.

In defining what the writer means, he quotes from his testimony in a recent case where a commission jointly estimated the cost of reproducing the physical property of a water-works, allowing all enhancement of value due to existing market and physical conditions, and deducting for depreciation. The element of value referred to in that line of testimony was defined as "going concern value."

"The method of computing 'going concern value' assumes that the knowledge of the general public with regard to the water service is fully developed, it assumes that the business of the existing plant, its patronage, etc., remains intact until delivered to the new plant. No longer time is needed for the recovery of business than is necessary to render the new property administratively and mechanically fit and serviceable for conducting the entire business of the old property, allowing proper time and capital for making the service connections, the getting of machinery into proper working order, elimination of all defects of construction, the duplication of the office records, and the perfection of the business and mechanical organization."

Nearly all the elements of value entering into the computation of the "going concern value," as above described, represents tangible property in one form or another, and may very properly embrace an additional element of operating capital. As thus described, the going value has no relation to the earnings of the property, the reproduction cost of which is being estimated. It may be computed as a percentage of the cost of reproducing the physical property, or in any similar manner.

As thus defined, the going value loses all connection and relation with so-called intangible value; in fact, many of the units entering into or going to make up this going concern value are susceptible to decay, as are other portions of the physical property, and, in rate-fixing cases, should have a percentage of earnings set aside to cover general maintenance as well as interest on the money thus invested.

In appraisals for purchase and sale, there is an additional element of value which may be properly considered, in order to compensate a successful and efficient management. Just what this increment of value should be can only be determined in specific instances, in the light of all the facts relating to the business organization and the progressiveness of the municipality furnishing the patronage for the public utility under appraisement. While this increment of value would not be considered in rate-fixing cases as part of the capital investment, the rates could be adjusted so as to support through surplus earnings an increment of value sufficient to invite investments in a public utility of the kind under appraisement, and to encourage intelligent and efficient management. A community which has had the benefit of such a management can well afford to pay something more, for a property which is well organized and well managed, than it would be willing to pay for one where the situation is reversed. Mr. Kiersted.

On the other hand, a mismanaged property, where earnings have been sufficient to pay a fair return on the invested capital and maintain the property in an efficient condition, but have been diverted from the natural and proper channels, should expect and should receive nothing short of heavy depreciation of value in an appraisal of the property, either for rate-fixing purposes or for purchase and sale. Such considerations as these, entering into the investigations leading to a final conclusion on the value of a public utility, as they properly should enter, involve the intelligent judgment of the appraiser, and force him out of the "strait-jacket" of reasoning through mathematical formulas or any set of rules based thereon.

It does not always follow, however, that the value, either for rate-fixing purposes or for purchase and sale, will be invariably equitable when computed on the basis of the cost of reproduction, for the simple reason that, in exceptional cases, it may give a value in excess of that at which a community can either afford to purchase, or support through its patronage. One instance of this kind in the writer's experience is of a town which retrograded, losing perhaps 40% of its population during the period in which the public utility had been in operation. In this instance there could be no question but what the public utility corporation would have to share in the shrinkage of values resulting from reduced patronage and the inability of the town to meet obligations which otherwise they could have met had the town continued to increase in population and taxable value at the same average rate as it did during its prosperous career.

In view of the many uncertainties surrounding the appraisal of public utilities, it would seem to be well-nigh impossible to formulate any set of rules to guide appraisers in estimating value except those which are of very general application and embrace the more fundamental principles relating to and governing work of this character.

Mr. Kiersted. The writer is not one who believes the Courts have led engineers into error in performing their part of the work relating to the appraisement of public utilities. The whole province of the engineer in a matter of this kind is to assist the Court by his evidence, and when engineers of wide experience, but representing opposing interests, present views widely at variance on matters regarding which they are supposed to have an intimate knowledge, the Court is certainly to be excused if the evidence is practically set aside or only partly considered. As a rule, the estimates of engineers of experience on the cost of reproducing a physical property should not and will not vary materially. In fact, a joint conference of engineers representing opposing parties will frequently result in a joint agreement as to values, except possibly on the question of the overhead charges which it is customary to allow in the appraisement of public utilities, and on questions involving going value—elements of value which can be settled quite as readily by the Court itself after hearing individual testimony. Joint agreement on most or all of the technical details, in advance of the hearing of evidence and the filing of a joint report as the evidence of all the engineers, would greatly abbreviate the work of all concerned, and perhaps simplify and materially assist the work of the Court itself, but, unfortunately, in most instances, counsel of the opposing parties do not sanction the joint conference.

One word more, in order to make more clear the writer's position with regard to capital investment: The earnings covering depreciation, in the sense in which the writer has used depreciation, are expected, among other things, to return to the investor the cost of abandoned property. Accordingly, as property is abandoned it should be charged off the capital investment account, and, in a similar manner, any unit substituted for an abandoned unit should be added to capital investment account. Thus the accounts would show the total investment as well as the active or present investment at any date of valuation.

Whatever of criticism there may be in these remarks attaches to the writer with perhaps even more force than to Mr. Grunsky, for the writer has used the sinking-fund method in computing depreciation in numerous instances, but believes that in it he has observed fallacies which should be eliminated, even though it involves the discarding of sinking-fund methods and the radical modification of past lines of procedure. He has nothing but thanks and congratulations to offer the author for presenting in such a comprehensive manner the results of his labors, and hopes that the issues which are thereby raised will be discussed fully and, finally, will result in simplifying and improving the methods used in the valuation of public service properties.

Mr. Grunsky. C. E. Grunsky, M. Am. Soc. C. E. (by letter).—The point which the writer particularly desired to emphasize in the paper was that when

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rates are to be fixed the investor in public service properties is entitled to adequate protection. The method of securing this protection intelligently was described, and the writer tried to make clear that there should always be an allowance in some form and under some name to compensate the owner of the public service property for hazard, and for management both during construction and under operation. This compensation should appear in the rate of interest to be earned rather than in the appraisal on which interest is to be allowed. Placing it in the appraisal is a convenient way of making the earnings, expressed in percentage thereof, appear low. In whatever way this may be handled, something more than ordinary interest on the properly invested capital should appear in the earnings, if these have been equitably fixed.

There is no intent to advocate the taking of any property without due compensation, as Mr. Higgins seems to infer from parts of the paper. When the broad principle is laid down therein that the owner of a public service property is entitled to be protected to the extent of his investment, first, in the matter of receiving thereon a proper return, and, second, in having the invested capital itself protected and ultimately returned, this is not to be understood in a strictly restrictive sense. There may be elements not covered by actual investment, which are, nevertheless, essential parts of the property and represent investment even though never actually paid for by the owner, and not appearing as having cost him anywhere near their market value. In such cases, however, it will generally be well to inquire into the circumstances attending their acquisition.

Water rights, such as Mr. Van Cleve cites, are to be considered as property which has value. As part of a public service property, they may or may not represent actual investment. Such water rights, nevertheless, are to be classed as elements to be included in the appraisal. Probably no hard-and-fast rule can be laid down for determining their value in the sense of what it might reasonably be assumed that they would cost if they had to be acquired from other owners. The circumstances attending the valuation, particularly of undeveloped water rights, are so varied, involving as they do all the uncertainties of present and future demand for power, that any satisfactory and conclusive suggestion along this line is hardly to be hoped for at this time.

In California the flowing water belongs to the people, but the riparian owner on a stream has certain paramount rights. A distinction is to be made, therefore, between a water right in the broad sense in which it is here generally referred to, when it is strictly a grant by the people on a par with a franchise, and, in the more restricted sense, comparable with the illustration used by Mr. Van Cleve, in which case the power right is the property of the riparian owner. In the one

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case the water right deserves the same treatment as a franchise; in the other case the fact that it has a market value is easily recognized, and its appraisal is a comparatively simple matter.

In reply to the question which Mr. Boucher asks concerning the appraisal of property such as a valuable right of way which cost little or nothing, the circumstances in each case should be taken into account. In the case which he cites, the railroad is not entitled, in strict equity, to a return on the full amount that the property would cost to-day, but only on a fair allowance for investment. In other words, there should be some limit to the right to a return on the unearned increment represented by the present high value of the right of way, if such value be measured by what it would cost if it had to be acquired to-day. This view appears to be in accord with the rulings of the Courts, which would give to the public service concern, practically as a bonus, the appreciation in the value of such properties. What shall be considered excessive appreciation, is a question which had best be answered only as special cases arise.

A similar question is that relating to the treatment of donations. Take, for example, a water company which is called on to extend its service into new territory not yet built on, the owners of which construct a distributing system of pipes and house connections and all that goes toward a satisfactory service and donate the same without cost to the water company. To add the cost of such property at once to the capital invested by the water company might work hardship and be unfair to the older consumers. When the new territory is developed, and the consumers, resident therein, take a fair share of the water, it may well be asked whether or not in fixing rates the fact should be taken into account that a part of the system was donated by the water users themselves. It would be equitable to deduct donations from the appraisal, allowing, however, adequately for the upkeep of the entire system, including depreciation. On the other hand, there is good basis for the view that it makes no difference how the property is acquired, and that the appraisers for rate-fixing purposes should ignore the fact that some of it may have cost nothing.

The difficulty that confronts the rate-fixing body in matters of this kind arises from the generally accepted theory of the past—as advanced by owners of public service properties—that they are entitled, as profit, to whatever they can get in the shape of bonus and also to the full amount of the unearned increment, represented by increased value of the elements which go to make up the whole of the property. In order to avoid controversy in the future, these matters should be made clear, and it is by the general recognition of fundamental principles that this can be brought about and misunderstandings avoided.

The question is asked by Mr. Coombs as to whether high dividends

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paid in the past should be taken into account in making appraisals. In a general way, yes. Consideration may be given to this fact just as well as to the fact that there have been lean years. Ordinarily high dividends would hardly be construed as repayment of invested capital, but when such dividends have been paid to the detriment of the property, when proper foresight has not been exercised in making provision for its expansion, when the requirement for deferred maintenance is high, it may be that such conditions are due in part to the high dividends that have been paid, and this is then a circumstance to be considered.

The method of dealing with depreciation, as indicated by Mr. Kiersted, is noted in the paper as applicable to any public service property in which the depreciation may fairly be considered equivalent to the average annual replacement requirement. This is set forth in Paragraph 14 of the fundamental principles and elsewhere. The bookkeeping in such a case would be as Mr. Kiersted describes it.

Such elements of value as those enumerated by Mr. Kiersted under his definition of "going concern" may properly be included in a valuation, and perhaps the designation, "going concern," is properly applied to them. Against their inclusion no protest is raised. The protest which is implied in the paper relates to the addition of intangible values, whether under this or other designations, to a demonstrable investment, or, as is more frequently the case, to an estimated value less depreciation, in order to bring the same up to what the appraiser thinks it ought to be. If there is to be any addition to the investment appraisal, let this be clearly stated, and it will make no difference under what designation it is added; or, as pointed out in the paper, let the appraisal be made without deduction of depreciation, and let the percentage rate of earnings be sufficiently high to do full justice to the owner of the property.

In closing, the writer wishes to express his appreciation of the reception which his paper has received and to thank those who have contributed to the discussion for emphasizing and making more clear some of the thoughts which were but imperfectly presented.

In this connection attention may be called to Mr. Kiersted's amplification of the writer's brief comment on the perpetual life of composite public service properties. This idea was incorporated by the writer, in 1901, in a report on the appraisal of the properties of the Spring Valley Water Company, which is the company supplying San Francisco with water. If properly applied the assumption of perpetual life will be found helpful in many cases where, for a plant taken as a whole, no fixed or definite term of life can be assumed.

In the case of a complex plant, such as was then under consideration, it was reasonable to fix its probable life at so long a period that it might be called perpetual. Parts of it would deteriorate

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This idea naturally resulted in the adoption of a method of calculation described in the paper as the method under which no deduction need be made for depreciation in valuing public service properties for rate-fixing purposes.

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TRANSACTIONS

Paper No. 1233

THE ANALYTICAL DETERMINATION OF THE DIMENSIONS OF THE GRAVITY RESISTING PARTS OF MASONRY DAMS.

By Maurice G. Parsons, Jun. Am. Soc. C. E.

The writer, desiring and being unable to find an analytical rather than graphical method of dimensioning multiple-arch dam buttresses, has deduced the formulas presented herewith.

These necessarily resemble, in general, the Wegmann formulas for gravity dams, as they fulfill the following conditions:

(1) No tension should exist.

(2) A certain fixed stress should never be exceeded, either on the up-stream or down-stream faces. and the down-stream maximum stress should be the lesser.

(3) Such economy of material as is consistent with safety should be realized.

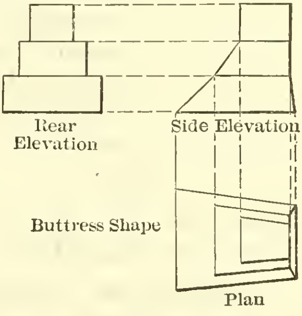

Fig. 1.

Nomenclature.—

Unit of weight = that of a cubic unit of masonry;

S = span of arches from center to center;

x = width of section wanted;

d = depth to Section x;

l = width of section next above x;

h = vertical distance between x and l;

B = thickness of Section x up stream;

b = " " " l up stream;

B' = thickness of Section x down stream;
b' = " " " l " ";
B'' = " " " x at toe of Section l;
a = top width;
w = weight of masonry above Section l;
W = " " " " " x;
n = distance from up-stream face to line of pressure, reservoir empty, for Section x;
u = distance from down-stream face to line of pressure, reservoir full, for Section x;
v = distance between lines of pressure for Section x;
m = n of Section l;
H = horizontal water thrust due to head, d_1, in masonry units;
p = maximum intensity of pressure, down stream;
q = " " " " up stream;
r = specific gravity of masonry;
y = offset from vertical plane through top of up-stream face;
M = the moment of the external forces about the heel;
R = reaction resulting from H and W.

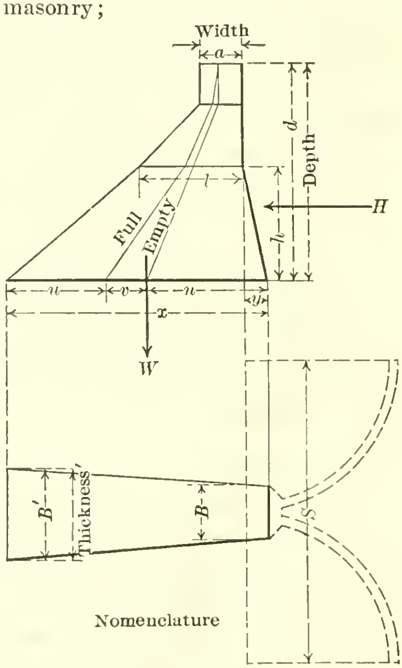

Fig. 2.

Case I.

Arches Less than Semicircles, but with Equal Radii.

Governing the derivations for buttress formulas are the limitations:

(1) The vertical component of the water pressure is neglected.

(2) A full head of water is assumed to act on each entire arch.

(3) The sides of the buttresses resemble steps with vertical risers and horizontal treads.

(4) Buttress plans may be trapezoids, but must be symmetrical, at any section, about a central axial plane common to all the sections of that buttress.

(5) Material giving gravity stability will be assumed to begin at the springing line of the arches, and its up-stream face shall be perpendicular to the axis of symmetry of the plan. The springing lines of various arch rings shall lie in a vertical plane. Masonry up stream from this plane will give greater stability against such a factor as sliding or neglected uplift. However, this up-stream buttress face is modified by an offset, y, below Section 2.

Section 1.—Starting with a top width, a, and thicknesses, b and b', how deep may we go before necessarily increasing the width?

We have,
$$x = u + v + n \qquad (1)$$
$$u = ? \qquad v = ? \qquad n = ?$$

$$M = \frac{H\ d}{3} = \frac{S\,d^3}{6\,r} = W\ v \qquad (2)$$

for v;
$$\text{therefore } v = \frac{M}{W} \qquad (3)$$

$$W = w + \frac{h}{4}\,(B + B')\,(l + x) \qquad (4)$$

Equations 1, 2, 3, and 4 are perfectly general.

For n:

n = distance from up-stream face to center of area of base

$$= \frac{a}{3}\,\frac{(2\,b' + b)}{b' + b} \qquad (5)$$

For u, see Fig. 3.

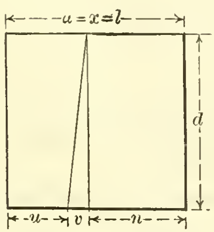

Fig. 3.

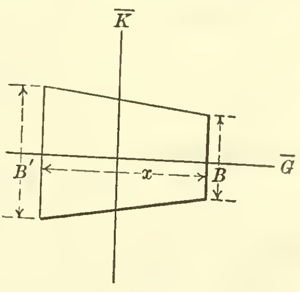

Fig. 4.

Referring to Fig. 4, with G and K as rectangular axes, having their intersection at the center of area of the base, we have:

$$h = \frac{W\,\overline{G}}{I_k}\,g + \frac{W\,\overline{K}}{I_g}\,k + \frac{W}{F} \qquad (6)$$

In which W = total pressure,

$(\bar{G}\ \bar{K})$ = point of application of W,

I = moment of inertia,

F = area,

and h = intensity of pressure at the point, $g\ k$.

What is $(\bar{G}\ \bar{K})$ to give $h = 0$ at the up-stream toe; that is, where is the line of pressure, reservoir full, for zero stress up stream?

From symmetry, $\bar{K} = 0$.

$$\text{Therefore } h = \frac{W\bar{G}}{I_k} g + \frac{W}{F} \qquad (7)$$

$$\text{Now } I_k = \frac{B'^2 + 4\,B\,B' + B^2}{36\,(B + B')} x^3 \qquad (8)$$

Putting $h = 0$, and g, designated by g'' = the distance to B from O, then $\bar{G}$ will give the distance from O to the line of pressure, reservoir full, for zero stress up stream.

$$g'' = \frac{x}{3}\,\frac{2\,B' + B}{B' + B} \qquad (9)$$

$$\text{then } 0 = \frac{W\bar{G}}{\frac{B'^2 + 4\,B\,B' + B^2}{36\,(B + B')} x^3}\,\frac{x}{3}\,\frac{2\,B' + B}{B' + B} + \frac{W}{\frac{B + B'}{2} x} \qquad (10)$$

$$\text{Solving Equation 10, } \bar{G} = -\frac{x}{6}\,\frac{B'^2 + 4\,B\,B' + B^2}{(B' + B)\,(2\,B' + B)} \qquad (11)$$

$$\text{The distance from } O \text{ to } B' = g' = -\frac{x}{3}\,\frac{B' + 2\,B}{B' + B} \qquad (12)$$

and u, for use until maximum down-stream pressure is reached,

$$= -g' + G \qquad (13)$$

$$= \frac{x}{3}\,\frac{B' + 2\,B}{B' + B} + G \qquad (14)$$

which, solved, gives:

$$u = -\frac{x}{6}\left(\frac{(B' + B)^2 + 2\,B\,B' - 2\,(B' + 2\,B)\,(2\,B' + B)}{(B' + B)\,(2\,B' + B)}\right) \qquad (15)$$

Putting the values of u, v, and n in Equation 1, and replacing x by a, B by b, and B' by b', we have:

$$x = a = -\frac{a}{6}\left(\frac{(b' + b)^2 + 2\,b\,b' - 2\,(b' + 2\,b)\,(2\,b' + b)}{(b' + b)\,(2\,b' + b)}\right) + \frac{S\,d^2}{3\,r\,(b + b')\,a} + \frac{a}{3}\,\frac{2\,b' + b}{b' + b} \qquad (16)$$

Reducing:
$$d = a\sqrt{\frac{3\,r}{S}\left(\frac{b' + 2\,b}{3} - \frac{(b' + b)^2}{2\,(2\,b' + b)}\right)} \quad \ldots\ldots\ldots\ldots \quad (17)$$
which is the required depth limiting Section 1.

Section 2.—After the line of pressure, reservoir full, has crept so far toward the toe that the pressure on the up-stream face = 0, the width of the base must be increased in order to avoid tension.

As in Section 1, $u = -\frac{x}{6}\left(\frac{(B'+B)^2+2\,B\,B'-2\,(B'+2\,B)\,(2\,B'+B)}{(B'+B)\,(2\,B'+B)}\right)$

$$\text{and } v = \frac{S\,d^3}{6\,r\left[w + \frac{h}{4}\,(B + B')\,(l + x)\right]}.$$

Section 2 may be considered as divided into: Section 1, with base A_1 and volume V_1; an oblique wedge, the base of which is A_2, and the volume V_2; and a triangular pyramid, the base of which is A_3 and the volume V_3.

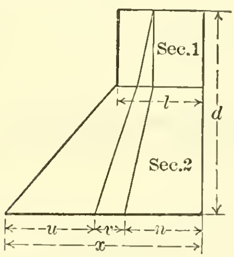

FIG. 5.

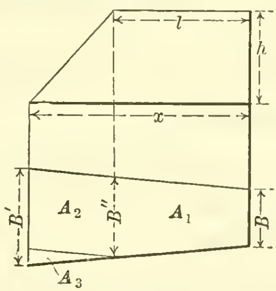

FIG. 6.

The distances of the centers of gravity of the various parts from the up-stream face are:

(1) For V_1 : $y_1 = \frac{l}{3} \cdot \frac{2\,B'' + B}{B'' + B} = q_1$.

(2) For V_2: The center of area of A_2 is distant horizontally from B'' an amount, $z_2 = \frac{x - 1}{2}$, and the center of gravity of the volume is distant horizontally from B'' an amount, $y_2 = \frac{x - 1}{3}$, and $q_2 = y_2 + l = \frac{x + 2\,l}{3}$.

(3) For V_3: The center of area of A_3 is distant horizontally from B'' an amount, $z_3 = \frac{2}{3}\,(x - l)$, and the center of volume of V_3 is distant horizontally from B'' an amount, $y_3 = \frac{x - l}{2}$, and $q_3 = y_3 + l = \frac{x + l}{2}$.

Taking moments about the up-stream vertical face:

$$V_1\, q_1 = \frac{h l^2}{6}\,(2\,B'' + B) \ldots\ldots\ldots (18)$$

$$V_2\, q_2 = \frac{B''\, h}{6}\,(x - l)\,(x + 2\,l) \ldots (19)$$

$$V_3\, q_3 = \frac{h}{12}\,(B - B'')\,(x^2 - l^2) \ldots (20)$$

$\overline{W} g$. in Fig. 7, $= V_1\, q_1 + V_2\, q_2 + V_3 q_3$. (21)

Taking moments about the up-stream face, Fig. 7:

$$W_n = w\, m + \overline{W}\, g \ldots\ldots\ldots (22)$$

But, $W = w + \dfrac{h}{4}\,(B + B')\,(l + x) \ldots (4)$

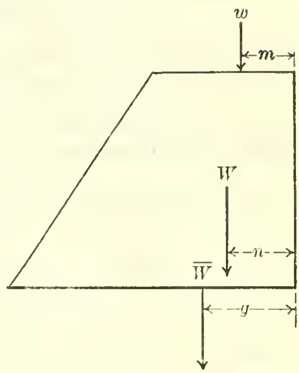

Fig. 7.

therefore,
$$n\, w + \frac{n\, h}{4}\,(B + B')\,(l + x) = w\, m + \frac{h\, l^2}{6}\,(2\,B'' + B) + \frac{B''\, h}{6}\,(x - l)\,(x + 2\,l) + \frac{h}{12}\,(B' - B'')\,(x^2 - l^2) \ldots\ldots\ldots\ldots (23)$$

From which,

$$n = wm + \frac{h}{6}\,\frac{\left(l^2 B + \frac{1}{2}\, B''\, x^2 + B''\, x\, l + \frac{1}{2}\, B'\, x^2 - \frac{1}{2}\, B'\, l^2 + \frac{1}{2}\, B''\, l^2\right)}{W} \ldots (24)$$

Now, again,

$$x = u + r + n \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

For x:

$$x = -\frac{x}{6}\left(\frac{(B' + B)^2 + 2\;B\;B' - 2\,(B' + 2\,B)\;(2\,B' + B)}{(B' + B)\;(2\;B' + B)}\right) + \frac{S\, d^3}{6\, r\left[w + \frac{h}{4}\,(B + B')\,(l + x)\right]}$$
$$+ \frac{w\, m + \frac{h}{6}\left(l^2\, B + \frac{B''\, x^2}{2} + B''\, x\, l + \frac{B'\, x^2}{2} - \frac{B'\, l^2}{2} + \frac{B''\, l^2}{2}\right)}{w + \frac{h}{4}\,(B + B')\,(l + x)} \ldots (25)$$

which reduces to

$$x^2\;\frac{h}{24}\left(\frac{5\,B'^2 + 10\;B\;B' + 3\;B^2 - 4\;B'\;B'' - 2\;B\;B''}{2\;B' + B}\right) + x\left[w + \frac{h\, l}{4}\,(B + B')\right]\left[1 - \frac{B' + B}{2\,(2\,B' + B)}\left(-\frac{h\, l\, B''}{6}\right)\right]$$
$$- \frac{S\, d^3}{6\, r} - w\, m - \frac{h\, l^2}{12}\,(2\;B - B' + B'') = 0 \ldots\ldots\ldots (26)$$

The value of x, as found by Equation 26, holds until the pressure on the down-stream face, reservoir empty, $= 0$.

For this limiting condition, taking (Equation 7) $h = \frac{W\,\overline{G}}{I_k} g + \frac{W}{F}$, putting $h = 0$, and $g =$ the distance from O to B', then $\overline{G}$ will give the distance from O to the lines of pressure, reservoir empty, for zero stress down stream. Under these conditions, Equation 7 becomes,

$$0 = \frac{W\,\overline{G}}{\frac{B'^2 + 4\,B\,B' + B^2}{36\,(B + B')}} x^3 \left[-\frac{x}{3} \frac{B' + 2\,B}{B' + B} \right] + \frac{W}{\frac{B + B'}{2}} x,$$

from which

$$\overline{G} = \frac{x}{6} \frac{(B' + B)^2 + 2\,B\,B'}{(B' + B)\ (B' + 2\,B)} \quad \text{(27)}$$

Now, $n =$ the distance from O to B, called g'' minus $\overline{G}$,

or
$$n = g'' - \overline{G} \quad \text{(28)}$$

$$g'' = \frac{x}{3} \frac{(2\,B' + B)}{B' + B} \quad \text{(9)}$$

Substituting and reducing:

$$n = \frac{x}{2} \frac{B' + B}{B' + 2\,B} \quad \text{(29)}$$

which is the condition limiting Section 2.

Section 3.—After n has reached a value of $\frac{x}{2} \frac{B' + B}{B' + 2\,B}$, less than which it cannot go without producing tension, we should, for economy of material, keep it there until another limiting condition arises. This may be done by increasing x and at the same time offsetting up stream so that n has its required value.

Again, $x = u + v + n$ .... (1)

$$u = \frac{x}{2} \frac{B' + B}{2\,B' + B}$$

$$v = \frac{M}{W}$$

$$n = \frac{x}{2} \frac{B' + B}{B' + 2\,B}$$

$$x = x \frac{(B' + B)}{2} \left[\frac{1}{2\,B' + B} + \frac{1}{B' + 2\,B} \right] + \frac{M}{W}.$$

which reduces to

$$\frac{x^2 h}{4}\left[(B+B') - \frac{3}{2}\frac{(B+B')^3}{(2B'+B)\ (B'+2\ B)}\right]$$

$$+ x\left[w + \frac{h\ l}{4}(B+B')\right]$$

$$\left[1 - \frac{3}{2}\frac{(B+B')^2}{(2\ B'+B)\ (B'+2\ B)}\right] - \frac{S\ d^3}{6\ r} = 0 \ldots\ldots\ldots\ldots (30)$$

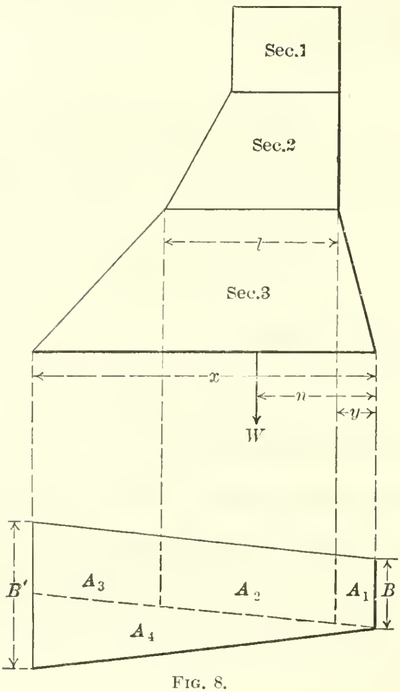

FIG. 8.

The required offset, y, may be found as follows:

Fig. 8 shows that any Section 3 may be divided into: a wedge, V_1, with base, A_1; a parallelopiped, V_2, with equal bases, A_2; a wedge, V_3, with base, A_3; and a wedge, V_4, with base, A_4.

The center of area of A_1 is distant, $\frac{y}{2}$ from B_1 and y_1; the distance from B to the center of gravity of $V_1 = \frac{2\ y}{3}$.

The center of gravity of V_2 is distant from B an amount, $y_2 = \frac{l}{2} + y$. The center of area of A_3 is distant from its up-stream face an amount, $\frac{x - l - y}{2}$, and the center of gravity of V_3 is distant from its up-stream face an amount, $\frac{x - l - y}{3}$, and from B a distance of $y_3 = \frac{x + 2\,l + 2\,y}{3}$.

The center of gravity of A_4 is distant from B, $y_4 = \frac{2\,x}{3}$.

Summarizing:

$$V_1 = \frac{B\,y\,h}{2} \qquad y_1 = \frac{2\,y}{3}$$

$$V_2 = B\,l\,h \qquad y_2 = \frac{l}{2} + y$$

$$V_3 = \frac{x - l - y}{2}\,h\,B \qquad y_3 = \frac{x + 2\,l + 2\,y}{3}$$

$$V_4 = \frac{x}{2}\,(B' - B)\,h \qquad y_4 = \frac{2\,x}{3}.$$

Taking moments about the heel line, B, and reducing, we get,

$$\frac{W\,n - w\,m - \frac{B\,h}{6}\,(x^2 + l\,x + l^2)}{w + \frac{B\,h}{6}\,(2\,l + x)} \qquad (31)$$

a general formula.

Section 3 holds until the limiting pressure, p, is reached. The expression for this, derived below, is

$$p = \frac{6\,W}{x\,(2\,B' + B)} \qquad (33)$$

Taking Equation 7 again, remembering what is known and that we want the toe pressure, reservoir full,

$$h = \frac{W\,\overline{G}}{I_k}\,g' + \frac{W}{F}$$

$$\overline{G} = -\frac{x}{6}\,\frac{(B' + B)^2 + 2\,B\,B'}{(B' + B)\,(2\,B' + B)} \qquad (11)$$

$$g' = -\frac{x}{3}\,\frac{B' + 2\,B}{B' + B} \qquad (12)$$

$$I_k = \frac{B'^2 + 4\,B\,B' + B^2}{36\,(B + B')}\,x^3 \qquad (8)$$

Substituting and reducing, the limiting down-stream pressure is

$$h = \frac{6\ W}{x\ (2\ B' + B)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (32)$$

Section 3, therefore, holds until

$$\frac{6\ W}{x\ (2\ B' + B)} = p \quad \ldots\ldots\ldots\ldots\ldots\ldots (33)$$

the assumed safe pressure on the down-stream face.

Section 4.—After $\frac{6\ W}{x\ (2\ B' + B)} = p$, this relation should be maintained, which can be done by increasing x so as to increase u the proper amount, leaving n as before.

For u: From Equation 7 we have, again,

$$h = \frac{W\ \overline{G}}{I_k} g' + \frac{W}{F}\text{, and want } u\text{, that is, } \overline{G} - G.$$

$$g' = -\frac{x}{3}\ \frac{B' + 2\ B}{B' + B} \quad \ldots\ldots\ldots\ldots\ldots\ldots (12)$$

$$I_k = \frac{B'^2 + 4\ B\ B' + B^2}{36\ (B + B')} x^3 \quad \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

$$\text{whence } h = \frac{W \overline{G}}{\dfrac{B'^2 + 4\ B\ B' + B^2}{36\ (B + B')}} x^3 \left(-\frac{x}{3}\ \frac{B' + 2\ B}{B' + B}\right) + \frac{W}{\dfrac{B + B'}{2}\ x}$$

$$\text{giving } \overline{G} = \left(\frac{2\ W}{x\ (B' + B)} - h\right)\left(\frac{B'^2 + 4\ B\ B' + B^2}{12\ W\ (B' + 2\ B)}\ x^2\right) \quad \ldots\ldots\ldots\ldots (34)$$

and a reduced value of

$$u = \left(\frac{x}{6\ (B' + B)} - \frac{p\ x^2}{12 W}\right) \left(\frac{B'^2 + 4\ B\ B' + B^2}{B' + 2\ B}\right) + \frac{x}{3}\ \frac{B' + 2\ B}{B' + B} . (35)$$

Substituting again in Equation 1 and reducing, n and v remaining the same as in Section 3, we get:

$$\frac{x^2\ p}{12\ W}\left(\frac{B'^2 + 4\ B\ B' + B^2}{B' + 2\ B}\right) = \frac{M}{W} + x\left(\frac{B'^2 + 3\ B\ B' + 2\ B^2}{(B' + B)\ (B' + 2\ B)} - 1\right) . (36)$$

the equation for determining x until the limiting up-stream pressure is reached.

For limiting conditions: Turning once more to Equation 7:

$$h = \frac{W\ G}{I_k}\ g'' + \frac{W}{F}$$

$$g'' = \text{distance from } O \text{ to } B = \frac{x}{3}\ \frac{2\ B' + B}{B' + B} \quad \ldots\ldots\ldots\ldots (9)$$

$$I_k = \frac{B'^2 + 4\ B\ B' + B^2}{36\ (B + B')}\ x^3 \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

$$G = \frac{x\ (B' + B)^2 + 2\ B\ B'}{6\ (B' + B)\ (B' + 2\ B)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (27)$$

Substituting and reducing, $h = \dfrac{6\,W}{x\,(B' + 2\,B)}$;

or, in other words, when $\dfrac{6\,W}{x\,(B' + 2\,B)} = q$ .... (37)

the assumed maximum up-stream pressure, Section 4 must be abandoned for Section 5.

Section 5.—By increasing x properly, p and q can be maintained at their assigned values indefinitely.

For n: Making final use of Equation 7:

$$h = \frac{W\,\overline{G}}{I_k'}\,g'' + \frac{W}{F}$$

$$g'' = \frac{x}{3}\,\frac{2\,B' + B}{B' + B} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

$$h = q$$

Substituting and solving,

$$\overline{G} = x^2\,\frac{B'^2 + 4\,B\,B' + B^2}{12\,W\,(2\,B' + B)}\left(q - \frac{2\,W}{x\,(B' + B)}\right) \ldots\ldots\ldots (38)$$

From the relation, $n = g'' - \overline{G}$, we get:

$$n = -\,x^2\,q\,\frac{B'^2 + 4\,B\,B' + B^2}{12\,W\,(2\,B' + B)} + x\,\frac{(2\,B' + B)}{3\,(B' + B)} + \frac{B'^2 + 4\,B'\,B + B^2}{6\,(B' + B)(2\,B' + B)} \ldots\ldots\ldots\ldots (39)$$

At last, using Equation 1, we arrive at

$$x^2\left[\frac{p}{B' + 2\,B} + \frac{q}{2\,B' + B}\right]\frac{B'^2 + 4\,B\,B' + B^2}{12\,W} = \frac{M}{W} + x\left[\frac{1}{B + 2\,B'} + \frac{1}{2\,B + B'}\right]\frac{B'^2 + 4\,B\,B' + B^2}{6\,(B' + B)} \ldots (40)$$

Discussion.—The foregoing formulas are not all perfectly general, that is, they cannot be applied to buttresses with an unsymmetrical plan. This general case, for use when the two arches coming into one buttress have unequal radii, and do not transmit parallel pressures on the buttress, and it is desired to keep the line of pressure due to either arch acting alone just at such a position on the base that the buttress will not upset, is complicated by the difficulty of finding the center of area of any unsymmetrical trapezoid and the center of gravity of the section built on it.

This same problem is implicated in determining a theoretical B', in the symmetrical case, of such magnitude that, should a single arch fail, its buttresses will just stand under the action of their other arches. In other words, to get a B' so that the line of pressure due to only one arch will cause a zero stress on one side of the buttress, it is necessary to be able to solve, for any trapezoid, Equation 6:

$$h = \frac{W\bar{G}}{I_k} g + \frac{W\bar{K}}{I_g} k + \frac{W}{F}.$$

Strictly speaking, B' and B'' are functions of B, a condition which will make Equation 26 one of the fourth degree, involving a cube in x, and will raise Equations 30, 36, and 40 to the third power of x.

Such a state of affairs would cause more trouble with an analytical than is given by the old cut-and-try graphical method. However, for all practical purposes, B, B', and B'' may be given actual values, thus making the formulas easy of solution. The magnitude of B is determined by the thickness of the arch ring and practical considerations of constructive features, while B' and B'', in turn, may be taken from B. Theoretically, the thickness of the arch ring and the value of B, if taken only large enough to accommodate the two arches, vary as a straight line, while the formulas are deduced for stepped values of B. However, having dimensioned the entire buttress, it may be changed so as to have regular and smooth sides by averaging the successive values of B, as in Fig. 9.

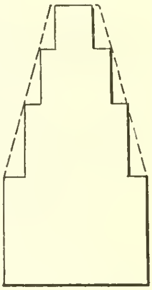

Fig. 9.

It may be remarked, further, that, as far as buttresses are concerned, arches need not be circular, but may be: First, of any shape as long as the two arches coming into any one buttress are symmetrical about that buttress; secondly, of any size as long as the lines of thrust of the arches coming into any one buttress are parallel.

Some engineers might always adopt a rectangular base, guided by such things as:

(1) Material gives a greater stability against overturning if placed up stream than if placed down stream.

(2) Greater lateral stiffness would be obtained by increasing B beyond its minimum value.

It may be well to note that Case I. and all cases shortly to be derived therefrom, are perfectly applicable to any liquid by using a proper value of r, namely,

$$r = \frac{\text{weight of a cubic unit of masonry,}}{\text{weight of a cubic unit of the liquid dammed.}}$$

Taking up now the limitations governing the derivation:

(1) Neglecting the vertical component of the fluid pressure on the face of the buttress may or may not give added safety, depending on whether the offset, y, extends or does not extend beyond the up-stream dotted line of Fig. 2 (plan).

(2) In some cases considerable material might be saved by computing the actual thrust, H, as modified by the rising ground under an arch.

(3) This has already been taken up in discussing the values of B; and (4) was considered under the perfectly general case.

(5) When testing graphically, the material up stream, not included in the formula, as, for example, arches and fillers or straight curtain-walls (as the case may be), might be considered, and the buttress trimmed down accordingly, or this may be used to counteract probable uplift under the buttress.

Case II.

If dealing with semicircular or semi-elliptical arches, the plan may be rectangular, in which event, $B = B' = B''$. Here, B can be with certainty chosen so that the line of pressure, should an arch go out, will lie within the central third of the buttress base.

Case III.

By considering S as the span from center to center of buttresses, instead of arches, we obtain formulas for dimensioning buttresses for dams with a straight water face. B, B', B'' may or may not be equal, but they have been in reducing the formulas, which, therefore, are identical with those of Case II.

Case IV.

The Wegmann formulas are arrived at from Case I by considering that $S = B = B' = B'' = 1$.

Case II was established prior to and independently of Case I, for immediate use at the time. Case I reduces to Case II as originally obtained, which, in turn, gives, in Case IV, by making the proper substitutions, the Wegmann formulas. Therefore, it has not been thought necessary to expend energy on graphical trials to verify the correctness of these formulas.

Résumé.—Case I.

Buttresses with Symmetrical Trapezoidal Bases.

General Formulas.—

$$x = u + v + n \quad \ldots\ldots (1)$$

$$M = \frac{H\,d}{3} = \frac{S\,d^3}{6\,r} = W\,v \quad \ldots\ldots (2)$$

$$v = \frac{M}{W} \quad \ldots\ldots (3)$$

$$W = w + \frac{h}{4}(B + B')\,(l + x) \quad \ldots\ldots (4)$$

$$y = \frac{W\,n - w\,m - \frac{B\,h}{6}(x^2 + l\,x + l^2) - \frac{x^2\,h}{3}(B' - B)}{w + \frac{B\,h}{6}(2\,l + x)} \quad \ldots (31)$$

Section 1.—

$$d = a\sqrt{\frac{3\,r}{S}\left(\frac{(b' + 2b)}{3} - \frac{(b' + b)^2}{2\,(2\,b' + b)}\right)} \quad \ldots\ldots (17)$$

$$n = \frac{a}{3}\,\frac{(2\,b' + b)}{b' + b} \quad \ldots\ldots (5)$$

$$u = \frac{x}{2}\,\frac{B' + B}{2\,B' + B} \quad \ldots\ldots (15)$$

Section 2.—

$$n = \frac{w\,m + \frac{h}{6}\left(l^2\,B + \frac{B''\,x^2}{2} + B''\,x\,l + \frac{B'\,x^2}{2} - \frac{B'\,l^2}{2} + \frac{B''\,l^2}{2}\right)}{W} \quad \ldots (29)$$

$$x^2\,\frac{h}{24}\,\frac{(5\,B'^2 + 10\,B\,B' + 3\,B^2 - 4\,B'\,B'' - 2\,B\,B'')}{2\,B' + B}$$

$$+ x\left[\left(w + \frac{h\,l}{4}(B + B')\right)\left(1 - \frac{B + B'}{2\,(2\,B' + B)}\right) - \frac{h\,l\,B''}{6}\right]$$

$$- \frac{S\,d^3}{6\,r} - w\,m - \frac{h\,l^2}{12}(2\,B - B' + B'') = 0 \quad \ldots\ldots (26)$$

$$u = \frac{x}{2}\left(\frac{B' + B}{2\,B' + B}\right) \quad \ldots\ldots (15)$$

This section holds until $n = \dfrac{x}{2}\,\dfrac{B' + B}{B' + 2\,B}$ ...... (29)

Section 3.—

$$n = \frac{x}{2} \frac{B' + B}{B' + 2\,B} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (29)$$

$$\frac{x^2\,h}{4} \left[(B + B') - \frac{3}{2} \frac{(B + B')^3}{(2B' + B)\,(B' + 2\,B)} \right]$$
$$+ x \left[w + \frac{h\,l}{4} (B + B') \right] \left[1 - \frac{3}{2} \frac{(B + B')^2}{(2\,B' + B)\,(B' + 2\,B)} \right]$$
$$- \frac{S\,d^3}{6\,r} = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots (30)$$

$$u = \frac{x}{2} \frac{B' + B}{2\,B' + B} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

A reduced value of y, as given by Equation 31, may be obtained to please the individual liking of the engineer.

Section 3 holds until $\frac{6\,W}{x\,(2\,B' + B)} = p \ldots\ldots\ldots\ldots\ldots\ldots\ldots (32)$

Section 4.—

$$u = -\,x^2\,p\,\frac{B'^2 + 4\,B\,B' + B^2}{12\,W\,(B' + 2\,B)} + x \left[\frac{B'^2 + 4\,B\,B' + B^2}{6\,(B' + B)\,(B' + 2\,B)} + \frac{B' + 2\,B}{3\,(B' + B)} \right] \ldots\ldots\ldots\ldots (35)$$

By Equation 29:

$$\frac{x^2\,p}{12\,W} \frac{(B'^2 + 4\,B\,B' + B^2)}{B' + 2\,B} = \frac{M}{W} + x \left[\frac{B'^2 + 3\,B\,B' + 2\,B^2}{(B' + B)\,(B' + 2\,B)} - 1 \right] \ldots (36)$$

The limiting condition is that

$$\frac{6\,W}{x\,(B' + 2\,B)} = q \ldots\ldots\ldots\ldots\ldots\ldots\ldots (37)$$

Section 5.—

$$x^2 \left(\frac{p}{B' + 2\,B} + \frac{q}{2\,B' + B} \right) \frac{B'^2 + 4\,B\,B' + B^2}{12\,W} = \frac{M}{W}$$
$$+ x \left(\frac{1}{B + 2\,B'} + \frac{1}{2\,B + B'} \right) \frac{B'^2 + 4\,BB' + B^2}{6\,(B' + B)} \ldots\ldots (40)$$

$$n = -\,x^2\,q\,\frac{B'^2 + 4\,BB' + B^2}{12\,W\,(2\,B' + B)} + x \left(\frac{2\,B' + B}{3\,B' + 3\,B} + \frac{B'^2 + 4\,BB' + B^2}{6\,(B' + B)(2\,B' + B)} \right) . (39)$$

$$u = -\,x^2\,p\,\frac{B'^2 + 4\,BB' + B^2}{12\,W\,(2\,B' + B)} + x \left(\frac{B'^2 + 4\,BB' + B^2}{6\,(B' + B)(B' + 2\,B)} + \frac{B' + 2\,B}{3(B' + B)} \right) . (35)$$

and the limiting circumstance is money.

Case II.

Buttresses with rectangular bases, used in particular with semicircular arches of equal radii, and also:

Case III.

Buttresses for plane surface curtain-wall water faces with $S =$ the distance from center to center of buttresses rectangular in plan.

General Formulas.—

$$v = \frac{M}{W} \ldots\ldots\ldots\ldots (3)$$

$$M = \frac{Sd^3}{6\ r} \ldots\ldots\ldots\ldots (2)$$

$$W = w + \frac{l + x}{2}\ h\ B \ldots\ldots\ldots\ldots (4)'$$

$$x = u + v + n \ldots\ldots\ldots\ldots (1)$$

$$y = \frac{Wn - wm + \frac{h\ B}{6}\ (l^2 + l\ x + x^2)}{w + \frac{h\ B}{6}\ (2\ l + x)} \ldots\ldots\ldots\ldots (31)'$$

Section 1.—

$$d = a\ \sqrt{\frac{B\ r}{s}} \ldots\ldots\ldots\ldots (17)'$$

$$n = \frac{x}{2} \ldots\ldots\ldots\ldots (5)'$$

$$u = \frac{x}{3} \ldots\ldots\ldots\ldots (15)'$$

Section 2.—

$$x^2 + \left(\frac{4\ w}{h\ B} + l\right) x - \frac{6}{h\ B}\ (M + w\ m) - l^2 = 0 \ldots\ldots (26)'$$

$$n = \frac{w\ m + \frac{h\ B}{6}\ (l^2 + l\ x + x^2)}{w + \frac{l + x}{2}\ h\ B} \ldots\ldots\ldots\ldots (24)'$$

$$u = \frac{x}{3} \ldots\ldots\ldots\ldots (15)'$$

The limiting condition is that $n = \frac{x}{3} \ldots\ldots\ldots\ldots (29)$

Section 3.—

After $n = \frac{x}{3}$, an offset, y, enters, given in value by .... (31)″

$$x^2 + \left(l + \frac{2\ w}{h\ B}\right)\ x = \frac{6\ M}{h\ B} \ldots\ldots\ldots\ldots (30)'$$

$$u = \frac{x}{3} \ldots\ldots\ldots\ldots (15)'$$

$$n = \frac{x}{3} \ldots\ldots\ldots\ldots (29)'$$

The limiting condition is that

$$\frac{2\ W}{B\ x} = p \ldots\ldots\ldots\ldots\ldots\ldots\ldots (32)'$$

Section 4.—

$$x^2 = \frac{6\ M}{p\ B} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (36)'$$

$$n = \frac{x}{3} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (29)'$$

$$u = \frac{2\ x}{3} - \frac{p\ x^2\ B}{6\ W} \ldots\ldots\ldots\ldots\ldots (35)'$$

The limiting condition is that

$$\frac{2\ W}{B\ x} = q \ldots\ldots\ldots\ldots\ldots\ldots\ldots (37)'$$

Section 5.—After $\frac{2\ W}{B\ x} = q,$

$$n = \frac{2\ x}{3} - \frac{q\ x^2}{6\ W} \ldots\ldots\ldots\ldots\ldots (39)'$$

$$u = \frac{2\ x}{3} - \frac{p\ x^2}{6\ W} \ldots\ldots\ldots\ldots\ldots (35)'$$

$$0 = x^2\ B\ (p + q - h) - x\ (2\ w + l\ h\ B) - 6\ M \ldots\ldots (40)'$$

Case IV.

By letting $S = B = B' = B'' = 1$, we get:

General Formulas.—

$$x = u + v + n \ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

$$v = \frac{M}{W} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

$$M = \frac{d^3}{6\ r} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)''$$

$$W = w + \frac{l + x}{2} h \ldots\ldots\ldots\ldots\ldots\ldots (9)''$$

$$y = \frac{W\ n - w\ m - \frac{h}{6}\ (l^2 + l\ x + x^2)}{w + \frac{h}{6}\ (2\ l + x)} \ldots\ldots\ldots (31)''$$

Section 1.—

$$d = a\sqrt{r} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (17)''$$

$$n = \frac{x}{2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)'$$

$$u = \frac{x}{3} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)'$$

Section 2.—

$$x^2 + \left(\frac{4\,w}{h} + l\right)x - \frac{6}{h}\,(M + w\,m) - l^2 = 0 \ldots\ldots \quad (26)''$$

$$n = \frac{w\,m + \dfrac{h}{6}\,(l^2 + l\,x + x^2)}{w + \dfrac{l + x}{2}\,h} \ldots\ldots \quad (24)''$$

$$u = \frac{x}{3} \ldots\ldots \quad (15)'$$

which holds until $$n = \frac{x}{3} \ldots\ldots \quad (29)$$

Section 3.—

$$x^2 + \left(\frac{2\,w}{h} + l\right)\,x - \frac{6\,M}{h} = 0 \ldots\ldots \quad (30)''$$

$$u = \frac{x}{3} \ldots\ldots \quad (15)'$$

$$n = \frac{x}{3} \ldots\ldots \quad (29)'$$

which holds until $$\frac{2\,W}{x} = p \ldots\ldots \quad (32)''$$

Section 4.—

$$x^2 = \frac{6\,M}{p} \ldots\ldots \quad (36)''$$

$$n = \frac{x}{3} \ldots\ldots \quad (29)'$$

$$u = \frac{2\,x}{3} - \frac{p\,x^2}{6\,W} \ldots\ldots \quad (35)''$$

The limiting condition is $$\frac{2\,W}{x} = q \ldots\ldots \quad (37)''$$

Section 5.—

$$x^2\,(p + q - h) - (2\,w + l\,h)\,x - 6\,M = 0 \ldots\ldots \quad (40)''$$

$$u = \frac{2\,x}{3} - \frac{b\,x^2}{6\,W} \ldots\ldots \quad (35)''$$

$$n = \frac{2\,x}{3} - \frac{q\,x^2}{6\,W} \ldots\ldots \quad (39)'$$

which are the familiar Wegmann formulas for gravity dams.

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TRANSACTIONS

Paper No. 1234

FAULTS IN THE THEORY OF FLEXURE, AND AN EPITOME OF CERTAIN I-BEAM TESTS MADE AT AMBRIDGE, PA.*

By Henry S. Prichard, M. Am. Soc. C. E.

With Discussion by Messrs. Charles Worthington, Edward Godfrey, S. Vilar y Boy, H. F. Dunham, Eugene W. Stern, Theodore Belzner, J. P. J. Williams, Edgar Marburg, and Henry S. Prichard.

As the ordinary theory of flexure is almost universally used, not only in proportioning simple beams, but in the solution of all questions involving the elastic deformation of structural members, the nature and influence of its faults, by reason of which it is not rigidly accurate, but only approximate, should be generally understood.

It is generally recognized that the ideal material and conditions assumed are not wholly achieved, and it is shown in some elaborate treatises on the theory of elasticity, but not ordinarily realized, that, even if it were possible to have ideal material and conditions, the theory would still be faulty; for instance, it is shown by Professor C. Bach,† that a cross-section originally plane does not remain plane during flexure, as is ordinarily assumed, but is forced into a reversed curve somewhat like a long $\int$, only much less pronounced in ordinary materials; and Professor A. E. H. Love‡ states that the ordinary equation for shear distribution gives an average intensity across the breadth of the section, and that actually the distribution is not uniform, as is tacitly assumed in nearly all textbooks.

* Presented at the meeting of May 1st, 1912.

† "Elastizität und Festigkeit," p. 459.

‡ "A Treatise on the Mathematical Theory of Elasticity," p. 331.

It is not necessary to master profound and highly complicated treatises on elasticity to understand the faults in the theory of flexure, while, for the purpose of allowing for these faults, after they are understood, judgment, assisted by approximations and tests, is of more practical use to engineers than the much involved expressions of mathematical investigators.

For convenience, the further discussion of the subject is divided into sections.

Section 1.—Deformation of Cross-Sections.

The fact that a cross-section originally plane is forced by flexure into a reversed curve, somewhat like a long $\int$, can be readily shown by marking the position of a cross-section on the sides of a free, good, soft, rubber eraser, such as is used by draftsmen, and then bending it by the thumbs and forefingers, or by loading it, as illustrated, Fig. 1 being the unloaded, and Fig. 2 the loaded, beam.

The curve developed in an originally plane cross-section by loading the beam can be explained by considering the distortion produced by shear. To simplify the analysis, consider a vertical cross-section of a horizontal beam at a point where there is no bending moment and where, consequently, the strains are due entirely to shear.

According to the theory of flexure, the shear will be greatest at the neutral axis and gradually decrease until it becomes zero at the extreme top and bottom fibers. The theory is correct in this regard, although faulty with reference to the law by which the shear diminishes.

The shear acting on the horizontal and cross-sectional faces of an originally square increment will cause one diagonal of the increment to lengthen and the other to shorten, as in the various increments shown in Fig. 3, and these distortions will be less for each succeeding increment from the neutral axis toward the top and bottom fibers. Consequently, the originally vertical transverse faces of these increments will not remain in the same transverse plane, but will form a curve, as in Fig. 3.

The curves of the successive cross-sections of a beam toward the point of no shear will gradually approach a straight line, and reverse in direction after the point of no shear is passed.

The intensity of the horizontal stresses in successive horizontal fibers will vary in accordance with the changes in the lengths of these

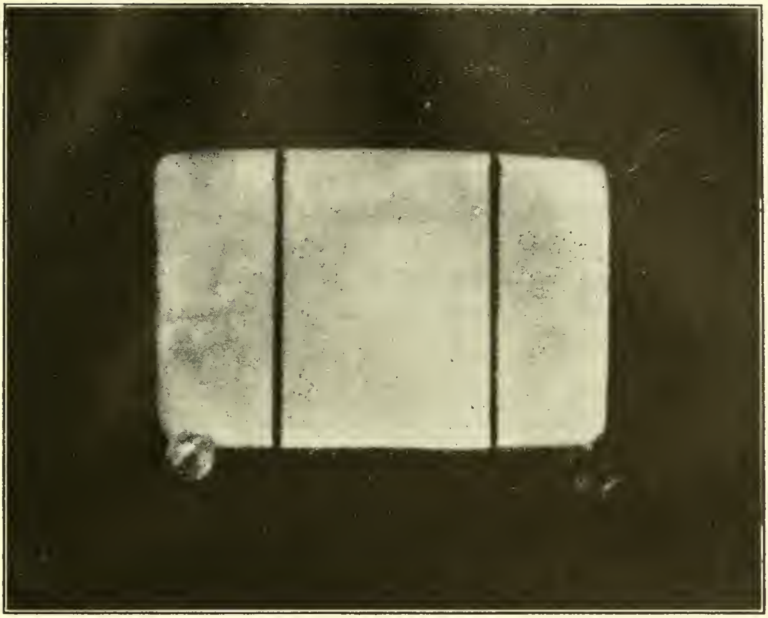

Fig. 1.—Rubber Beam: Not Loaded.

Fig. 2.—Rubber Beam: Loaded.

fibers, but these changes, evidently, will not be in direct proportion to the distances of the fibers from the neutral axis, as indicated by the ordinary theory of flexure; hence the ordinary equations for determining the extreme fiber stresses, in which the moment of inertia is a factor in the amount of the stress, are not strictly accurate, because this use of the moment of inertia is based on the proposition that the intensity of the stress in any fiber varies in direct proportion to its distance from the neutral axis.

In these circumstances the questions arise: "Can the theory be corrected in this regard?" and "To what does the error involved amount in practice?"

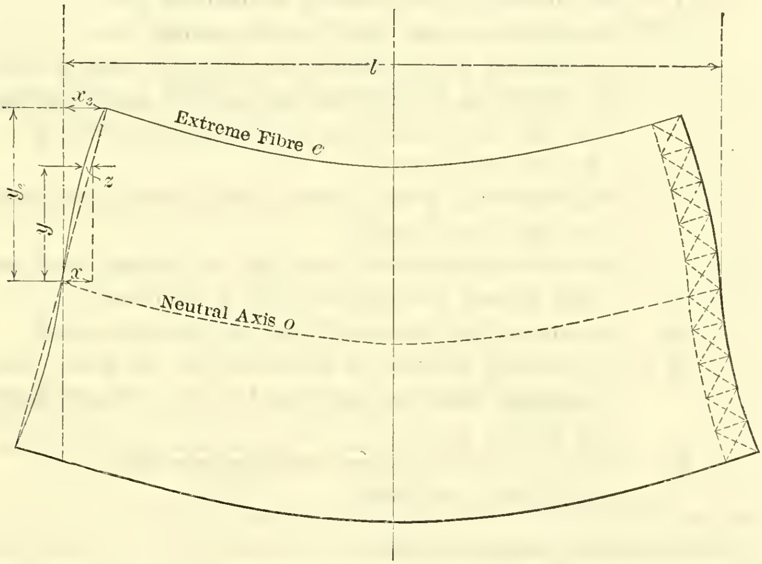

Fig. 3.

An exact general analysis, if developed, would be so complicated, and its application would necessitate so much labor and consume so much time, that it would be wholly impracticable to put it to any general use. It is practicable, however, to determine, in cases selected as criteria, close approximations to the corrections which should be made to allow for the error involved in assuming that cross-sections originally plane remain plane during flexure.

For horizontal beams of constant and usual cross-sections, uniformly loaded within the elastic limit, and with ends simply supported, the cross-sections at the center will remain vertical during flexure (as is evident from the symmetry of the condition as regards the center), the maximum intensity of stress in each horizontal fiber or layer will occur at the center cross-section, and the change in the lengths of the various fibers will be proportional to the maximum intensity of stress therein.

The change, caused by flexure, in the length of each fiber from the end to the center of the beam, is also the amount by which the end of each fiber moved (from its original free position in the vertical plane passing through the end of the neutral axis), as shown in Fig. 3.

Let f = the intensity of the stress in the extreme fiber,
y = the distance of any fiber from the neutral axis,
y_e = the distance of the extreme fiber from the neutral axis,
x = the shortening of any fiber between the center and end of the beam, indicated by the ordinary theory of flexure for a given f,
x_e = the shortening of the extreme fiber between the center and end for a given f,
z = the difference between x and the true shortening of any fiber between the center and end of the beam,
a' = the area of any horizontal layer of the cross-section,
M = the bending moment at the center of the beam corresponding with f, as determined by the ordinary theory of flexure,
M' = the true bending moment corresponding with f,
l = the length of the beam.

By the ordinary theory of flexure:

The intensity of the stress in any fiber is

$$\frac{f\,x}{x_e} = \frac{f\,y}{y_e}$$

For any given f, M is constant for all values of l, and

$$M = \sum_o^e \left(\frac{f\,a'\,x\,y}{x_e} = \frac{f\,a'\,y^2}{y_e}\right) = \frac{f\,I}{y_e} \quad \ldots\ldots\ldots\ldots (1)$$

By a refined method:

$$M' = \sum_o^e \left(\frac{f\,a'\,x\,y}{x_e} - \frac{f\,a'\,z\,y}{x_e}\right) \quad \ldots\ldots\ldots\ldots (2)$$

The mean intensity of stress in the extreme fiber between the center and the end of the beam equals $\frac{2f}{3}$

$$x_e = \frac{f\,l}{3\,E} \text{ . . . . } (3)$$

From Equations 1 and 3, Equation 2 becomes

$$M' = \frac{f\,I}{y_e} - \left(3\ E \sum_o^e a\ z\ y\right) \div l \text{ . . . . } (4)$$

As a study for a contemplated paper on plate-girder design, the writer made a comparison, which is given in Table 1, between the results of Equations 1 and 4 for the steel beams shown in Figs. 4A, 4B, 4C, 4D, and 4E.

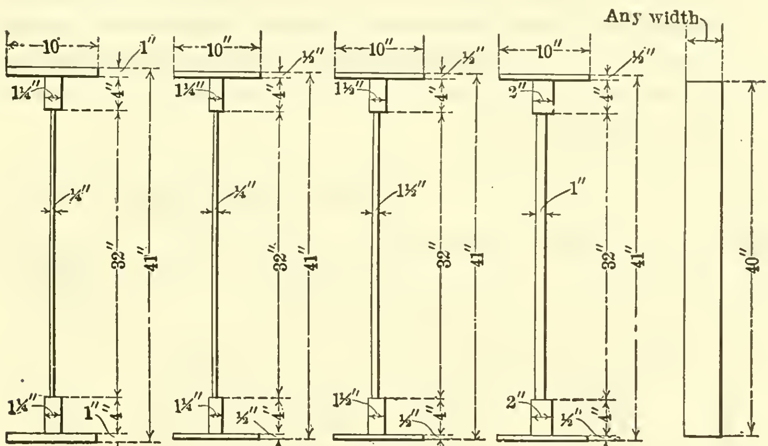

FIG. 4A. FIG. 4B. FIG. 4C. FIG. 4D. FIG. 4E.

In computing the numerical value of the true bending moment, the values of z were determined for the distribution of shear indicated by the ordinary theory of flexure, which distribution thus applied tends toward a slight under-estimate of the value of the bending moment; the lateral contraction in the web accompanying and at right angles to the extension from tension was taken as one-third of the extension, and the lateral extension accompanying and at right angles to the contraction from compression was taken as one-third of the contraction, which ratio is, if anything, somewhat greater than the mean of experiments; and the summation in the second member of Equation 4 was rendered simple and closely approximate by the homely device of dividing the beam into a considerable number of finite elements and considering the mean shear in each as the average of the extremes, which tends toward a slight over-estimate of the value of the bending moment. The net result of these approximations is to over-state slightly the error involved in the assumption that originally plane cross-sections remain plane during flexure.

In determining the maximum length for which shear is the governing consideration, the greatest permissible intensity in shear was taken as three-fourths of that in tension.

TABLE 1.—Giving, for the Beams Shown in Figs. 4A, 4B, 4C, 4D, and 4E, the Percentages by which the Indicated Capacity for Uniformly Distributed Load, when Computed by the Ordinary Theory of Flexure, Using the Extreme Fiber Stress as the Criterion, Should be Reduced to Allow for the Error Involved in Assuming that Originally Plane Cross-Sections Remain Plane During Flexure.

Beam in figure.	Ratio of web area to total area.	Coefficient.	Percentages by which Indicated Capacity should be Reduced. For Length ÷ Depth, as below.		Percentages by which Indicated Capacity should be Reduced. For Length ÷ Depth equals 10.
(1)	(2)	(3)	(4)		(5)
4A	28 to 100	219 400	332″ ÷ 42″ = 8.0	2.0	1.24
4B	36 to 100	153 700	234″ ÷ 41″ = 5.7	2.8	0.91
4C	54 to 100	122 700	134″ ÷ 41″ = 3.3	6.0	0.73
4D	71 to 100	94 000	88″ ÷ 41″ = 2.1	11.0	0.56
4E	100 to 100	81 100	40″ ÷ 40″ = 1.0	37.0	0.51

For girders 4A, 4B, 4C, and 4D, shear governs when length is less than given in Column 4.

When the lengths of the above girders are more than twice their depths, the approximate percentages of reduction can be obtained by dividing the coefficients given in Column 3 by the squares of their lengths, in inches.

In obtaining the ratios given in Column 2, the web was taken the full depth of the beam.

The beams from which Table 1 was computed have thin webs, but the webs can be increased without affecting the results, provided corresponding changes are made in the flanges.

A consideration of Table 1 shows that for very short beams the erroneous assumption that originally plane cross-sections remain plane during flexure leads to a considerable over-estimate of their capacity to resist bending stresses, while for long beams and those of moderate length the error is of little practical importance.

Section 2.—Manner of Loading.

Beams frequently rest on supports, and occasionally are suspended; loads are applied sometimes at the top and sometimes at the bottom; and, in the case of I-shaped beams, the loads and reactions are sometimes distributed as nearly as practicable over the entire depth of the web.

A fault, and, as far as concerns I-shaped beams with thin webs, the most serious fault, in the theory of flexure is that it does not take into account the manner in which beams are loaded and supported, but is developed on the tacit assumption that just the right proportion of each load and reaction needed to produce the theoretical changes in shear reach each horizontal layer of the beam without producing any stress in the layers above or below.

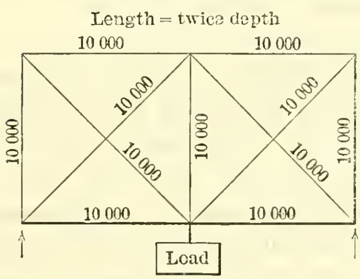

Fig. 5A.

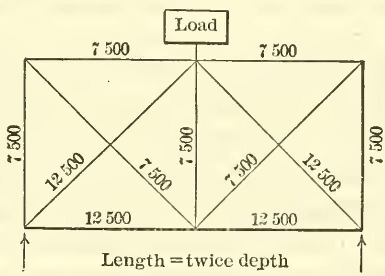

Fig. 5B.

When this tacit assumption is not realized, the distribution of shear and horizontal stress will not be the same as indicated by theory, and vertical tension, or compression, as the case may be, will be produced in the web.

A double lattice girder with a length of twice its depth, shown in Figs. 5A and 5B, is chosen to illustrate by analogy the principles involved, because it is a simple case in which the stresses can be readily determined from the laws of elasticity.

In such a lattice girder the changes in the stress in the diagonals occur at the top and bottom, and if the loads and reactions are applied in suitable proportions at these points there will be no strain in the vertical members, in fact, no need for vertical members; but, if otherwise applied, vertical members will be strained.

In the case illustrated in Fig. 5A the loads and reactions are applied entirely at the bottom and the vertical members are therefore strained. The girder is designed so that the stresses in each member have an intensity of 10 000 lb. per sq. in. If, now, the position of the load is changed from the bottom to the top, as in Fig. 5B, more members will be strained in one system than in the other; therefore, it will take less load to produce the common deflection in one system than in the other, and the stresses will be less in one than in the other. In fact, the stresses will be increased in the members of one system and decreased in those of the other by 25 per cent.

By analogy, it is proper to infer that similar differences occur in the distribution of shear and horizontal stresses in beams, and should be considered, when the beams are very short, in gauging their capacity. The percentage of difference rapidly decreases with increase in length, and is inconsiderable in beams of ordinary lengths.

It is usual and necessary in designing built I-beams, known as plate girders, to provide for the vertical compression in the webs, from heavy concentrated loads and reactions, by reinforcing the webs with vertical stiffeners between the flanges. As is well known, it is not customary to do this with rolled I-beams. The only other way of avoiding the overstraining of the webs, in such cases, is to use I-beams in which the webs and flanges are proportioned so that there is sufficient metal in the webs to resist, not only the shear indicated by the ordinary theory of flexure, but, in addition, the tendency of loads applied at the top and reactions applied at the bottom to crush and buckle them.

Architects and engineers should give earnest attention to this phase of the subject. The old and tried shapes, which for many years have been standard for I-beams, have fairly thick webs and well and amply proportioned connections between the webs and flanges; but new shapes, made possible by new methods of rolling, are now rolled which have a greater proportion of metal in the flanges, and for which greater strength in proportion to their weight has been computed by the ordinary theory of flexure and unreservedly claimed, but which have webs in which resistance to crushing and buckling under concentrated loads and reactions has been considerably reduced, as compared with the resistance of the webs in the old shapes.

Section 3.—Distribution of Shear.

The ordinary equation for distribution of shear, criticized by Professor Love, is as follows:

Let Q = the total shear on any cross-section of a beam of constant cross-section.

q = the intensity of the shear at any point in the cross-section. (See text below and conclusion at end of this section.)

m = the statical moment of that portion of the cross-section outside of the horizontal line in which intensity of the shear is obtained, taken about the neutral axis.

b = breadth of the cross-section at the point where the intensity of the shear is obtained.

I = the moment of inertia of the entire cross-section.

$$q = \frac{Q\,m}{I\,b} \qquad (5)$$

This equation is usually given as applicable to solid sections of beams of all possible shapes. Except for the influence of the faults discussed in Sections 1 and 2, it really gives, as pointed out by Professor Love, the mean or average shear across the breadth of the cross-section. The tacit assumption, in most of the textbooks, that the intensity of the shear is uniform across the breadth of the cross-section, can be analyzed.

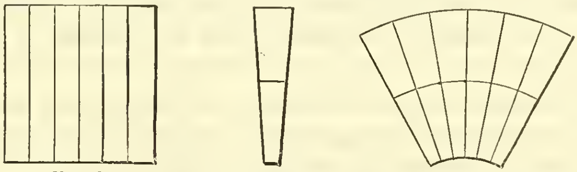

Fig. 6A. Fig. 6B. Fig. 6C.

If a number of very thin, independent, equal, rectangular beams are placed side by side, as in Fig. 6A, and then loaded, the portion of each in compression will be laterally expanded and the portion of each in tension laterally contracted, as in Fig. 6B; and, if the loads on each are suitably varied, by increasing them from the center toward the outside beams so as to produce the necessary deflections, and if the sides of the beams are brought into contact, they will collectively appear as one beam with a cross-section bounded on the top and bottom by curved lines and on the sides by lines inclined toward each other, as in Fig. 6C.

The elemental beams, on account of their extreme thinness, have no lateral stiffness, and can be brought into contact by lateral forces so small that the stresses they produce are negligible.

If, without disturbing the position or shape of the elemental beams, their sides are now joined so that the hitherto separate beams form a single homogeneous beam, of which they are equal vertical layers, there will be no stress or shear on their vertical sides, but each layer will be in the same condition of stress and shear as it was when an independent beam; and the shear on the combined beam will not be uniform across the cross-section, but will increase from the center outward.

If, after joining the original elements, the load on the intermediate vertical layers is increased to equal the load on the outside layers, each intermediate layer will deflect, but, in so doing, will transmit part of its load to the adjacent layer toward the outside. The outside layers, therefore, will continue to carry more than a *pro rata* share of the total load, and therefore have more than a mean intensity of shear.

For very broad, very shallow rectangular beams, such as could be formed by a wide, thin plate, the difference in distribution of shear across the breadth of the cross-section is considerable, but, for ordinary rectangular cross-sections, it is evident that the lateral deformation affects the deflection of the different vertical layers so little, in comparison with the total deflection, that there will be hardly any appreciable variation in the shear across the breadth of the cross-section.

These conclusions agree with those of St. Venant, who was the first to make a satisfactory mathematical investigation, and his conclusions were endorsed by Sir William Thomson (Lord Kelvin).*

The influence of lateral deformation on deflection, and, consequently, on distribution of shear, will similarly be of little consequence in solid beams with round, oblong, diamond, or other symmetrical cross-sections, which are not unduly broad and gradually reduce in breadth from the neutral axis toward the extreme fibers, as in Figs. 7A, 7B, 7C, and 7D.

If the distribution of shear in such a beam was analogous to the distribution in a large number of very thin independent vertical beams

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*Encyclopædia Britannica (Ninth Edition), Vol. VII, p. 809.

having the same deflection, and, in the aggregate, the same cross-section as the beam under consideration, the load carried by, and, consequently, the shear on any cross-section of, any one of the vertical layers, as compared with the entire beam, would, unless the beam was very short, be closely proportional to their respective moments of inertia, and the mean intensity of shear would be closely proportional to their moments of inertia divided by their areas; that is, to the square of their radii of gyration. (This proposition is based on the ordinary equations for deflection, with the qualifying word "closely" added on account of the faults discussed in Sections 1 and 2, and of the omission from the ordinary equations of the influence of shear on deflection.) Further, cross-sections of the vertical layers, being rectangular, would, according to Equation 5, have a maximum intensity of shear exceeding the mean intensity in the proportion of 3 to 2. Applying these propositions to the center vertical layer:

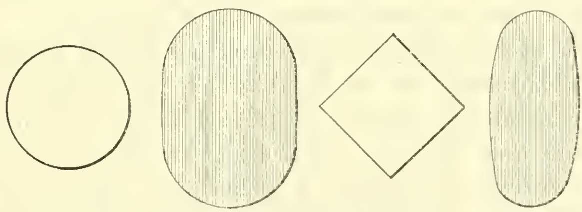

FIG. 7A. FIG. 7B. FIG. 7C. FIG. 7D.

Let r = the radius of gyration of the entire cross-section,
and h = the depth of the center vertical layer,

The square of the radius of gyration of the center vertical layer $= \frac{h^2}{12}$.

The ratio of the mean intensity of shear in the center vertical layer to the mean intensity on the entire cross-section is as $\frac{h^2}{12}$ is to r^2 ....(6)

And the ratio of maximum intensity of shear to mean intensity of shear on the entire cross-section, is as $\frac{h}{8}$ is to r ....(7)

Table 2 is a comparison of ratios of maximum to mean intensity for various cross-sections, as derived by applying Equations 5 and 7, respectively.

TABLE 2.

Cross-section.	By ordinary Equation, 5.	By Equation 7.
Rectangular	3 to 2	3 to 2
Round	4 to 3	2 to 1
Square diamond	1 to 1	3 to 1

The assumption on which Equation 7 is based, that the vertical layers act like independent beams having a common deflection, is not tenable, however, as the deformation from shear, illustrated in Fig. 3, in adjacent independent beams would not match, but would be greatest toward the center. In the united section each successive vertical layer, from the center toward the outside, in distorting would transmit some of its shear to the adjacent section. Hence the ratio of maximum to mean intensity of shear would be intermediary between the values indicated by Equations 5 and 7.

For the square diamond there is another method of determining the maximum shear, the results of which are suggestive. If the load is resolved into components parallel to the directions of the sides of the beam, and if the intensity of the shear from each component is derived by Equation 5 (which is a close approximation for a square cross-section with the load thus applied), and if the maximum intensities of the shear from each component are combined, the ratio of maximum to mean thus obtained is 3 to 2, which is the same as for rectangular cross-sections, and probably not far off for any of the cross-sections in this class.

The distribution of shear in beams with solid rectangular, round, oblong, and diamond, cross-sections is of academic rather than of practical interest, as shear is not a critical matter in such beams unless they are very short, in which case, owing to the faults discussed in Sections 1 and 2, the ordinary theory of flexure is too faulty to use, and experiments should be the criteria.

In giving the ordinary equation (Equation 5), textbooks should state that q is the mean intensity of shear across the breadth of the cross-section at any point, and that, for rectangular cross-sections and webs of I- and T- beams, the intensity of the shear is nearly uniform across their breadth, but that it varies for other forms of cross-sections.

Section 4.—Buckling of Webs and Flanges.

The ordinary theory of flexure, besides being in some respects faulty, is incomplete in that it does not indicate the buckling which under certain conditions takes place in the webs and top flanges of certain types of beams.

This phase of the subject is susceptible of further analysis, but the methods of dealing with it will probably always remain somewhat empirical.

When the compression flange is supported at such intervals that the compression is nearly constant from point to point, the practice of limiting the intensity of the compression to that allowed by good column practice cannot be much in error. When the compression flange of a beam simply resting on end supports is laterally supported only at the ends, it differs from a column in having the compression increase toward the point of maximum from zero at the ends, instead of being constant throughout. Under these conditions, the tendency to lateral deflection is somewhat less than it would be if the flange was a column. In usual cases, for beams of constant cross-section, the length of the equivalent column might be taken as 10% less than the length of the beam, between end lateral supports, without undue risk.

In deep beams with thin webs the tendency to buckle is not confined to inclined and vertical directions, nor does it occur only at the ends and points where the loads and reactions are concentrated. There is likewise a tendency to buckle in a horizontal direction from direct compression at points between the neutral axis and the compression flange, and this tendency increases toward the point of maximum bending moment. The tendency of compression, in lines inclined at 45°, to buckle the web is offset, in part at least, by the contra tendency of tension at right angles thereto to take out buckles.

At points where the web is stiffened to resist concentrated loads and reactions, the stiffeners, if properly arranged, receive direct compression, but at other points the function of stiffeners is, as their name implies, simply to stiffen, that is, to increase the resistance of the web against lateral deflection.

The reinforcement of the webs to prevent crushing and buckling at points of concentrated loads and reactions is sometimes advisable in rolled beams, especially in some of the recent shapes referred to in Section 2, but the stiffening at other points, while often necessary in built beams, is hardly likely to be needed in rolled ones, as the ratio of web thickness to depth is probably sufficient in beams of the proportions thus far rolled, to avoid this necessity.

Built beams or plate girders, as they are usually termed, have so many special points that a discussion of their details is reserved for a separate paper.

Section 5.—Faulty Application of Theory.

In addition to making provision in designing for faults and omissions in the ordinary theory of flexure, engineers should guard against faulty application of the theory. It is a common practice to use a single unsymmetrical section, such as an angle or channel, as a beam, and to compute its nominal strength by the theory of flexure. Actually, the theory of flexure does not give the stress in such beams. Take a channel, for instance; if loads and reactions are applied in the plane of the web, as in Fig. 8A, the flanges receive their stresses from the web eccentrically, the intensity of the stresses is correspondingly increased, and the channel is warped in deflecting; while, if the loads and reactions are applied at the center of gravity of the channel, as in Fig. 8B, there is a tendency to bend the web, and develop serious complex stresses, in addition to those computed from the ordinary theory.

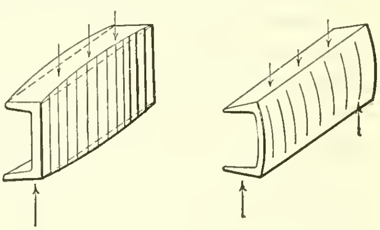

Fig. 8A. Fig. 8B.

It is generally best to avoid the use of unsymmetrical sections as beams unless connected in symmetrical pairs or otherwise laterally supported. When they are used it should be with a liberal allowance.

In ordinary practice, the stresses in beams are computed only for direct stress in the extreme fiber and shearing stress at the neutral axis; yet, according to the theory of flexure, the critical points in beams under concentrated loads may lie between the neutral axis and the extreme fiber. As an illustration, consider the case of the 30-in. girder beam, Fig. 9, under a load of 439 000 lb. concentrated at the center of a simple span of 79.64 in.:

In these conditions, the shear per square inch at the neutral axis is 12 000 lb. and the extreme fiber stress is 16 000 lb. per sq. in., but the direct stresses at the foot of the fillet, 2.4 in. from the top of the beam, are: compression 18 660 lb. per sq. in., and tension, at right angles to the compression, 5 220 lb. per sq. in. (and, *vice versa,* 2.4 in. from the bottom of the beam). If the ratio of lateral compression to longitudinal extension is one-third, these compound stresses will produce the same linear compression as would be caused by a simple compressive stress of 20 400 lb. per sq. in., and any shear of more than 142 300 lb., at a cross-section where the extreme fiber stress is 16 000 lb. per sq. in., will produce linear strains greater than would be produced by a simple stress of 16 000 lb. per sq. in.

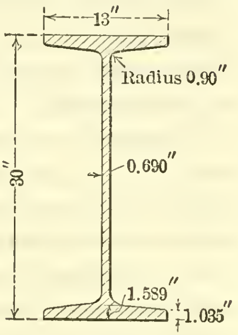

Fig. 9.

In making these computations, I was found to be 8 194.5; m, at neutral axis = 309.12; m, at foot of fillet = 254.35; and q, at foot of fillet = 9 874 lb. per sq. in. (when Q is 219 500 lb.) and 6 400 lb. per sq. in. (when Q is 142 300 lb.).

The maximum direct stresses at foot of fillet were found by the usual equation:

$$\text{Maximum direct stress} = \frac{1}{2} f_y \pm \sqrt{q^2 + \frac{1}{4} f_y^2} \qquad \text{.......... (8)}$$

in which f_y is the horizontal stress at the point where the direct stresses are required.

Section 6.—Overstrained Beams.

The theory of flexure, even after allowing for its faults, is only strictly applicable within the elastic limit of the material.

The elastic limit, even for specimens from the same melt of steel, will vary greatly, according to the amount of work put on them in rolling; and the original elastic limit, that is, the point where there will be a slight permanent set, is likely to be very low on the first application of the load. There is, however, a point in wrought iron and in soft and medium steel (known as the yield point, and often called the elastic limit), which is well marked in direct tension and compression tests, at which the metal, which before has shown only slight imperfections in elasticity, begins to flow rapidly.

Many experiments have shown that imperfections in elasticity, indicated at stress intensities below the yield point in iron and steel strained to the yield point, disappear, after a rest, on subsequent applications of the load, the explanation being that the original imperfections were caused by initial internal stresses which were removed by overstraining.

There has been much confusion with regard to the elastic limit, and it is not possible to tell from some reports of tests of beams whether the elastic limit recorded was simply an imperfection which the first loading would correct, or whether it was the critical elastic limit. It would be well in all doubtful cases, after beams under test loads have shown some permanent set and before testing them to destruction, to have the loads removed and the beams retested, after a rest.

Solid sections, such as pins, can, according to the theory which considers the effect of overstraining, develop, with a slight and almost inappreciable permanent set, a considerable permanent strength in excess of that indicated by the ordinary theory of flexure. If a horizontal pin without internal stresses is strained to the elastic limit by a vertical load, the intensity of the stresses decreases almost uniformly from the outer fibers to the center, but if the load is increased, the overstrained fibers toward the top and bottom deform so easily, as compared with the others, that, instead of the stresses decreasing uniformly from the extreme top and bottom toward the center, the metal for quite a distance from the top and bottom, if the load is sufficient, will be strained to the elastic limit; thus greatly increasing the capacity of the pin in the direction of the load. If the load is gradually taken off, the fibers toward the top and bottom will be entirely relieved of their stress before those nearer the center, after which tension will be developed in the top and compression in the bottom, forming a couple balanced by compression between the top and the center and tension between the bottom and center; further, the pin will have a permanent deflection. If the load is again applied, the effect of taking off the load will be reversed, without any additional overstraining, unless the original load is exceeded. If the direction of the load is reversed, the internal stresses will tend to lower the elastic limit of the pin.

There is another element which tends to enhance the permanent strength of overstrained solid beams like pins: When iron or steel is overstrained it becomes plastic, but resolidifies when the load is removed. On the removal of the load, the change during a rest from a plastic to a solid state, at a temperature much below the solidifying point, has an effect somewhat analogous to that of sudden cooling on soft and medium steel; it causes the metal to have a finer grain and a higher elastic limit.

Some experiments by Professor Thurston* on 1-in. square wrought-iron beams, 22 in. between supports, and loaded in the center, well illustrate the elevation of the elastic limit from overstraining. One of these beams showed some loss of elasticity under a load of 203 lb. and an extreme fiber stress of 6 700 lb. per sq. in.; yet it subsequently developed, as nearly as could be measured, seemingly perfect elasticity under a load more than eleven times as great.

It may be inferred that overstrained I-beams, especially those in which the metal has not been spread out too thin in the effort to obtain a large moment of inertia, will similarly develop considerable permanent elevation of the elastic limit, provided they are proportioned and laterally supported so that they will not buckle; but suitable tests are needed before this can be regarded as a certainty.

I-beams are peculiarly susceptible to initial internal stresses, and, therefore, to imperfections in elasticity within the yield point, as the flanges, being thicker than the webs, are yet hot after the webs have cooled and in cooling compress the webs horizontally and are themselves brought into tension. If the upper and lower halves of a beam were independent tees, they would bend in cooling so that the flanges would be on the insides of opposite curves, but, being joined, they are prevented from curving and, instead, develop in the web vertical tension at the ends and vertical compression at the center. In the days of wrought-iron beams it was not uncommon to have their webs split horizontally at the ends from such tension.

Conclusion.

The ordinary theory of flexure was gradually developed by noted scientists, beginning with Galileo, and was finally put on a solid mathematical basis by Navier, in 1824. While it is faulty and incomplete,

*Report of the U. S. Board for Testing Iron, Steel, etc., 1881, Vol. 1, pp. 455–472.

it is, considering the intricacy of the problems with which it deals, a remarkable approximation, and, when used in the light of reason, an excellent guide within wide limits.

Within the elastic limit, its faults, as applied to well-proportioned and well-supported beams, are practically important only for very short ones; which, unfortunately, have less theoretical resistance within the elastic limit than indicated by the ordinary theory.

The theory assumes that loads and reactions will be applied over the full depth of the beam, and that the profile of the beam and lateral supports are such that it will not buckle or develop weakness locally and will not buckle laterally; but the theory does not show how to insure these conditions, nor does it indicate the modification in the strength of the beam when they are not realized.

In trying to reconcile the theory with facts, the additional difficulties arise: that material has some imperfections in elasticity under stresses much less than what is ordinarily understood as the elastic limit, that wrought iron and soft and medium steel can have their elasticity perfected and its limit elevated by overstraining, and that overstraining introduces internal stresses in beams by which a greater proportion of the strength of the material is utilized under subsequent loads in the same direction (provided there has been no permanent buckling or serious injury).

In addition to the uncertainties incident to the faults in and limitations to the ordinary theory of flexure, there are uncertainties as to the effects of various methods and conditions of manufacture, on beams of various size and profile, which lie entirely outside the scope of the questions dealt with by the ordinary theory and can only be settled by scientific experiments.

The practical man, professedly skeptical in regard to theories, finally adopted the theory of flexure as a criterion for the strength of rolled I-beams (influenced, no doubt, by the statement, in a pioneer manufacturer's pocket book,* of the favorable results of actual tests, made at Trenton, of iron I-beams, by a United States Government engineer), and placed such confidence in its results that, when steel was substituted for wrought iron, and new shapes of I-beams were devised, their strength was assumed from theory, without tests; and when, within the last few years, new methods of rolling made it possible to roll

* New Jersey Steel and Iron Company's "Book of Useful Information," p. 33.

deeper beams, wider flanges, and thinner webs, the ordinary theory of flexure was still relied on as a sufficient criterion of the strength of the new shapes adopted.

The changes which have been made in the profiles of I-beams are quite marked, as shown in Figs. 10A, 10B, and 10C.

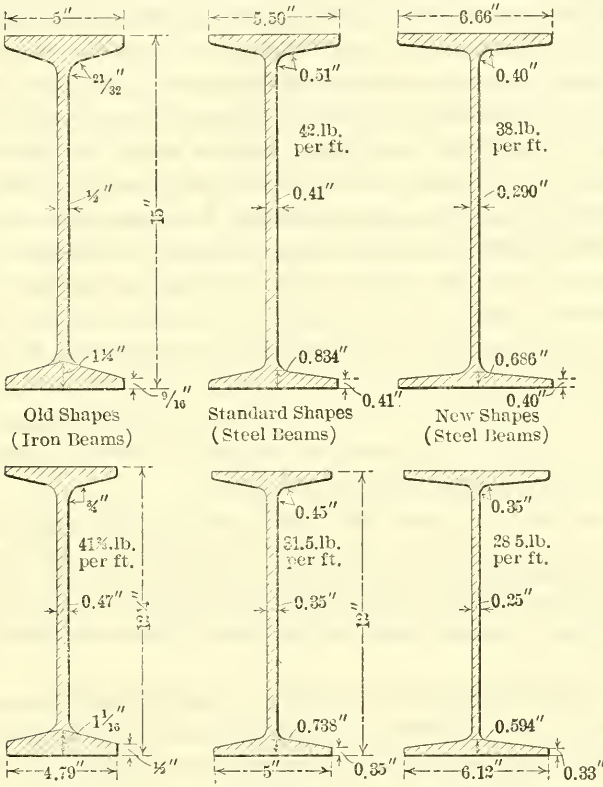

FIG. 10A. FIG. 10B. FIG. 10C.

The more the centers of gravity of the flanges are moved toward the top and bottom, by making the flanges wider and thinner, the greater the computed resistance to bending in proportion to the area of the cross-section; yet there must be some limit beyond which the metal is actually rendered less effective by such spreading and thinning, and this limit can only be determined by the behavior of beams in service and by scientific experiments.

Since the introduction of new shapes for steel beams, 31 beams have been tested by Edgar Marburg, M. Am. Soc. C. E., and a large number by certain manufacturers for their information and guidance.

In Professor Marburg's tests,* some indicated very low elastic limits, especially for the deeper beams, the lowest being 10 800 lb. per sq. in. for a 30-in. girder beam.

These low elastic limits have caused apprehension in the minds of Professor Marburg and other engineers. That the real original elastic limit, however, as distinguished from the yield point, is likely to be very low has long been known. About 74 years ago Mr. Eaton Hodgkinson found that any stress, however small, was sufficient to produce a set in cast-iron beams;† some 30 years ago the U. S. Board, in making bending tests on wrought-iron I-beams, found the elastic limit as low as 13 000 lb. per sq. in.;‡ and numerous tests at the Watertown Arsenal show low elastic limits for steel of excellent quality (for instance, a test of an eye-bar for the late George S. Morison, Past-President, Am. Soc. C. E., showed a permanent set at 5 000 lb. per sq. in.§).

Professor James Thomson, in 1848, before there were any retests of material to guide him, explained low original elastic limits as the result of initial internal stresses, and stated:

"It appears to me that the defects which he [Hodgkinson] has shown to occur even with very slight strains, exist only when the strain is applied for the first time, or, in other words, that if a beam has already been subjected to a considerable strain, it may again be subject to any smaller strain in the same direction without taking a permanent set."||

This remarkable prediction has been supported by subsequent experiments, the most notable of which are those by Professor Johann Bauschinger, described in his "Communications, 1886," and referred to by Professor Marburg, who stated as follows:

"Accordingly, after an initial stress, of a given magnitude within the elastic limit, has been once developed, the material is afterward perfectly elastic up to the limit of that stress."¶

It is much to be regretted that some of the I-beams tested by Professor Marburg and others were not experimented with, after they

* In a paper by him in *Proceedings*, Am. Soc. for Testing Materials, Vol. IX, 1909, p. 378.
† Report of the British Association for 1837, p. 362.
‡ Report of U. S. Board on Testing Wrought Iron, Steel, etc., 1881, Vol. 11, p. 226.
§ Report for 1901, p. 410.
|| Cambridge and Dublin Mathematical Journal, 1848.
¶ *Transactions*, Am. Soc. C. E., Vol. XLI, p. 227.

had some appreciable but not injurious permanent set, in order to ascertain the effect of overstraining on elasticity. It would be well to have some experiments in which the load would be removed and, after a rest, gradually re-applied, and the elasticity carefully observed and recorded, and others which would develop the greatest load under which, if allowed to remain indefinitely, the deflection would not be excessive and would finally cease to increase.

In Professor Marburg's tests the beams simply rested on supports, and concentrated loads were applied on the top flanges, which had no lateral support even at the ends, a severe combination of conditions, rarely encountered, which cannot be regarded as good practice.

The most extensive of the manufacturer's tests previously referred to were made, under various conditions of loading, on 12-in. and 15-in. steel I-beams.

The conditions of loading and supports were as follows:

A With end connection angles and loads applied at top,

B With end connection angles and loads applied by connection angles through the web,

C Supported on seat angles with loads applied at top,

D Supported on seat angles with loads applied by connection angles through the web.

The beams were tested for all four of these conditions, with loads applied at the center of the span, and also with loads applied at the third points of the span; that is, (1) with one load, and (2) with two loads.

The tests embraced beams of standard shapes and of new shapes; the averages of the preliminary specimen tests are given in Table 3.

TABLE 3.

	Flange Values.		Web Values.	
	Ultimate tensile strength, in pounds per square inch.	Yield point, in pounds per square inch.	Ultimate tensile strength, in pounds per square inch.	Yield point, in pounds per square inch.
Standard Shapes:				
15-in.–42 lb.	62 166	38 211	60 800	39 141
12-in.–31.5 lb.	62 834	39 222	61 076	38 266
New Shapes:				
15-in.–38 lb.	61 811	40 056	64 768	43 255
12-in.–28.5 lb.	60 588	41 026	64 184	49 647

The bending tests were made at Ambridge, Pa., and the construction of the machine necessitated the placing of the beams in a horizontal position, but they were guided and supported against lateral deflection at intervals of one-third their length.

Table 4, giving the loads which caused permanent sets of 0.1 and 0.4 in. in 15-in. 42-lb. per ft. and 12-in. 31.5-lb. per ft., I-beams of standard shapes, as in Fig. 10B, and 15-in. 38-lb. per ft. and 12-in. 28.5-lb. per ft. I-beams of new shapes, similar to those in Fig. 10C, was compiled from the manufacturer's diagrams of permanent sets.

The loads are stated in terms of the working load, W, computed for the nominal shapes of the beams, as given in manufacturers' pocket-books and shown in Figs. 10B and 10C, on the basis of 16 000 lb. per sq. in. in the extreme fiber.

TABLE 4.—LOADS WHICH PRODUCED PERMANENT SETS OF 0.1 AND 0.4 IN. IN BENDING TESTS OF 15-IN. 42-LB. AND 12-IN. 31.5-LB. I-BEAMS OF STANDARD SHAPES, AND 15-IN. 38-LB. AND 12-IN. 28.5-LB. I-BEAMS OF NEW SHAPES.

Depth of beam, in inches.	Span, in feet.	Loading, as explained above.	Working load; W, in pounds.	PERMANENT SET 0.1 IN.		PERMANENT SET 0.4 IN.	
				Standard shapes.	New shapes.	Standard shapes.	New shapes.
15.....	21	(1) A	14 980	3.73 W	3.20 W	4.07 W	3.73 W
15.....	21	(1) B	14 980	4.00 W	*3.35 W	4.27 W	*3.92 W
15.....	21	(1) C	14 980	3.64 W	3.15 W	4.19 W	3.83 W
15.....	21	(1) D	14 980	4.03 W	*3.54 W	4.15 W	*4.07 W
15.....	21	(2) A	22 470	3.43 W	3.16 W	3.83 W	3.36 W
15.....	21	(2) B	22 470	2.84 W	2.64 W	3.68 W	3.29 W
15.....	21	(2) C	22 470	3.53 W	2.82 W	3.89 W	3.34 W
15.....	21	(2) D	22 470	2.88 W	2.68 W	3.47 W	3.29 W
12.....	16	(1) A	12 000	3.87 W	2.92 W	4.12 W	3.85 W
12.....	16	(1) B	12 000	*4.04 W	*3.48 W	*4.21 W	*4.30 W
12.....	16	(1) C	12 000	3.92 W	3.22 W	4.33 W	4.21 W
12.....	16	(1) D	12 000	*4.22 W	*3.86 W	*4.50 W	*4.50 W
12.....	16	(2) A	18 000	3.32 W	2.66 W	3.78 W	3.24 W
12.....	16	(2) B	18 000	3.35 W	2.57 W	3.80 W	3.35 W
12.....	16	(2) C	18 000	3.58 W	2.73 W	3.89 W	3.51 W
12.....	16	(2) D	18 000	3.59 W	2.91 W	3.89 W	3.53 W

* Single tests: In all other cases the averages of three tests are given.

The actual dimensions of the beams of the new shapes that were tested were somewhat different from the nominal dimensions, as their webs were thicker and their flanges thinner, and their actual moments of inertia, and, therefore, their theoretical capacities, were about 5%

less than those of the standard beams of which they are nominally the theoretical equivalents.

It appears, from an examination of Table 4, that the beams sustained for short periods loads more than three times the working load without acquiring permanent sets large enough to be serious when viewed merely as changes in shape. Slight permanent sets, even under the working loads, would not in themselves be objectionable, and would not be alarming if it could be shown that permanent or indefinitely repeated loads of, say, twice the working loads could not produce failure or serious deformation.

On an average, it took 18.6% more load to produce a permanent set of 0.1 in. in the beams of standard shape than in the nominally equivalent beams of new shapes, and 8% more to produce a permanent set of 0.4 in. Whether or not this indicates a corresponding superiority in permanent capacity, what the permanent capacities are, and what permanent sets the beams would take under their maximum permanent loads, are questions to be decided by scientific experiments.

DISCUSSION

Mr. Worthington.

CHARLES WORTHINGTON, M. AM. SOC. C. E. (by letter).—The writer is of the opinion that this paper shows more faults in the application of the theory of flexure than it does in the theory itself.

The illustration of the rubber beam, Figs. 1 and 2, is quite misleading. In Fig. 1 the author has taken an ordinary velvet rubber and marked two parallel lines on it at about the quarter points of its length. Then in Fig. 2 he shows the effect on these lines of hanging a load by a string thrown over the center of the rubber while it is supported at the two lower ends after the manner of a beam. From the text, in the first paragraph of the author's Section 1, he would lead us to believe that this illustration is evidence that beams generally are affected in the same way under load.

Now, the writer would suggest that the author take away the end supports shown in Fig. 1, substitute a support continuous over the entire bottom of the rubber, and then apply the center load just as he has done in Fig. 2. The top ends of the two lines will come together just about as they did in Fig. 2, and without any beam action whatsoever. Then, by reversing the continuous support and placing it at the top, the outward bends of these lines at the bottom could likewise be obtained without any beam action. These ʃ-shaped curves are produced by local distortion, and not by bending. There is just about as much similarity between this rubber beam under the load to which it is subjected by the author, and that of an actual beam of steel, as there is between cheese under the action of a knife and a steel girder web in shear. In both the rubber and the cheese, the material is so soft that almost any applied load affects it locally from the outset.

The author's Section 2 seems to be merely an invidious comparison between the "old and tried shapes which for many years have been standard for I-beams," and the beams, rolled by the Bethlehem Steel Company exclusively, which are designated by the author as "new shapes."

The writer can see no reason, in theory or in fact, for making any distinction between the old or standard shapes and the new or Bethlehem shapes in regard to the effect of applying concentrated loads. The designer should always consider the matter of how each concentrated load shall be applied to the beam he is designing, whatever its section. It is only in extraordinary cases that buckling under concentrated loads other than end reactions will affect the design of any beam, and it is only in very extraordinary cases that crushing of the web need be considered at all. So it would seem to be a very poor argument to keep the webs of all beams thicker than necessary for nearly every purpose in order to provide for occasional concentrated loads which can be better and more effectively transferred to the beams by stiffeners milled to bear against the flanges and riveted directly to the webs of the beam, than by simply resting the concentrated load on top of the beam flange. As for the concentrated loads represented by the end reactions, these should, in all the larger sections, be taken care of by end stiffeners or, where they are framed into other beams, their equivalent in the form of connection angles. Mr. Worthington.

The author starts his Section 6, Overstrained Beams, with the paragraph:

"The theory of flexure, even after allowing for its faults, is only strictly applicable within the elastic limit of the material."

As a matter of fact, the theory of flexure is not at all quantitatively applicable to an overstrained beam as a whole. The theory of flexure is founded on the fact that strain and stress vary together, but as soon as the beam is overstrained, the stress in the extreme fibers passes the elastic limit of the material, and thereafter the strain in these fibers does not vary in proportion to the stress, so that one of the fundamental conditions of the theory is broken up, and, of course, the theory is not applicable thereafter to that beam.

Just as soon as the extreme fibers of a beam are stressed beyond the elastic limit of the material, the beam has failed and is no longer useful in the art of construction—the engineer's vital interest in the beam ceases when the elastic limit has been reached in any of its parts, and after that he wants to know only how long the member will stand up under its load until replaced. Yet even in the case of the overstrained beam, the ordinary theory of flexure indicates what will happen after the theory itself ceases to apply to the beam as a whole. Assuming that the beam load is being gradually applied, the stress in each longitudinal fiber increases from the start in the same ratio as the load, while the actual stress in any fiber is proportional to its distance from the neutral axis, the extreme fibers being stressed greatest of all. As the load continues to increase, the stress in the extreme fibers will in time reach the elastic limit of the material, and these fibers will then fail.

If the fibers that have failed were removed, the remaining portion of the beam would again act in accordance with the theory of flexure until its extreme fibers were in turn stressed beyond the elastic limit, and so on.

If the overstrained fibers are left in place, as is the usual case, the understressed fibers will tend to conform to the theory of flexure, but the result will be modified by the action of the overstrained fibers in offering some resistance to bending, though to a smaller and smaller extent, until the beam actually breaks. As soon as the outer

Mr. Worthington. fibers become overstrained, that is, when they become subjected to stress greater than the elastic limit, they will not recover completely on release of the load, and so the beam will take a permanent set or deflection.

Now, in comparing the effects of overstraining on two beams of the same depth and different cross-sections, it will be seen that the beam with the greater proportion of extreme fibers in its total area should receive the greater injury, as measured by the amount of permanent set, when the elastic limit of the beam is first passed; but as the loading increases for each beam until the entire flange is overstrained, this difference should diminish and possibly become reversed, for the beam with the greater flange area would, if anything, require a greater load for actual final rupture than the other. After the flanges of both beams become overstrained throughout their area, they are on practically the same footing, so that the resistance of the beam to final rupture resolves itself, excepting for the small effect of the web, into the relative strength of the flanges, and the one with the greater flange area should win out in the race.

This is what the theory of flexure indicates, and this is what Table 4 proves, if it proves anything. This table shows that the standard beams take permanent sets of 0.1 in. under an average load some 18.6% greater than the Bethlehem beams of equal rated strength, the loading at this point being for all cases greater than that which the theory of flexure indicates as being the elastic limit of the beams. This permanent set is but a small part of what the total deflection would be under such a loading, so it might fairly be taken to represent damage done to the extreme fibers only of the beams just after the elastic limit of the beam is passed. In which case the loads producing a given amount of set in the two types of beam should be inversely proportional to the amount of extreme fiber in each, or, in other words, to the flange widths.

Comparing the flange width of the Bethlehem beams of Fig. 10C with the flange width of the standard beams of Fig. 10B, it will be seen that for the 15-in. beams the former is 21% greater than the latter, and for the 12-in. beams, 22% greater; showing that the author's 18.6% of difference in loading necessary to produce the initial set of 0.1 in. in the two types of beam is so close to the theoretical amount as to be quite remarkable. Certainly such close agreement between theory and tests should bar out this table from a paper intended to demonstrate faults in that theory.

Table 4 shows further that, as the loading is increased in the tests recorded by this table until the permanent set is 0.4 in., the average load on the standard beams is only 8% greater than on the Bethlehem beams. Here the tests stop abruptly, and there is nothing to show how much farther this difference diminishes.

Now, the writer would call attention to the fact that the average load producing the permanent set of 0.4 in. is, for the standard beam, 21% greater than the average load producing the permanent set of 0.1 in., while, for the Bethlehem beam, it is only 11% greater. Mr. Worthington.

In other words, between these two points of the test, the standard beam is approaching ultimate destruction just about twice as fast as the corresponding Bethlehem beam, and at the last point of the record they are only 8% apart, according to the author's own statement. At this rate, the author's 18.6% of implied advantage for the standard beam, just after the elastic limit is passed, soon vanishes, and if this rate were maintained a very little longer in the test, the Bethlehem beam will show even a longer endurance than the standard shapes. After all, the real measure of the comparative value of the two beams, after their elastic limit has been passed, is, not how much deflection a given load will produce in each, but how long they will last under a continually increasing load. The beams have already failed as permanent structures when their elastic limit is passed, and after that the important thing to know is their comparative loads at the point of final rupture, their permanent set being of no real consequence excepting as an index to the loading that has been sustained.

It must be borne in mind all the time that these new or Bethlehem shapes require 10% less metal than the standard shapes with which these direct comparisons are being made.

In reference to Table 4, the writer would like to inquire in passing just how the permanent sets recorded in that table were determined so closely. In the process of applying the test load to any elastic structure, there is always some elastic recovery after the load is removed or released, and yet, if this table records actual measurements, it indicates that the operator knew in advance exactly how much this elastic recovery would be in each case, and applied a test load of such amount that after the elastic recovery of the beam had taken place, just exactly the permanent set of 0.1 or 0.4 in. was left.

This table records some 84 full-sized tests, of 44 standard and 40 Bethlehem beams, on a comparative basis, and yet not one single phenomenon of real value to the engineer in determining the relative merits of the two types of beam is included. It is incorporated in a paper, the avowed purpose of which is to point out and demonstrate to fellow engineers faults in the theory of flexure, and yet nearly all the measurements that would have been of value to the members of the Profession in judging the author's assertions and conclusions are carefully omitted in the record of these tests. Why are not the deflections under varying load recorded? These are capable of being determined by the theory of flexure, and the actual amounts under test could have been easily measured for comparison with the figures predetermined by the theory of flexure.

Mr. Worthington.

Why were not the strains measured on the extreme fibers of the beams? These strains are capable of being determined by the theory of flexure, and could easily have been measured and recorded for comparison with those predetermined by calculations.

Why were not the elastic limits of the beams under load recorded? These, too, were of prime importance, and could have been noted and recorded with great ease.

Finally, why were not the ultimate loads noted and recorded?

Surely these important measurements must have been considered of sufficient value to note down in passing when the comparatively valueless permanent sets were so accurately determined.

Either those who made the tests recorded in Table 4 were incapable of appreciating the relative importance of the phenomena that were thus allowed to pass by without note, or else the record of these phenomena was purposely withheld from the table—for surely it is not a tenable hypothesis that the author did not appreciate the overwhelming importance to his paper of including all these measurements in his record of those tests.

The whole gist of this paper is the comparative value of the standard and Bethlehem beams, and the diversions from this theme are few and unimportant. Someone has even gone to the trouble and expense of making 84 full-sized tests, on beams of equal rating for the two types, beam for beam, to fortify some opinion on the question of their relative values to the Engineering Profession, and yet the author, acting as the medium for presenting the records of those tests to this Society, has presented them in such shape that the vitals are all lacking, and has not even given any explanation of why this was done.

After describing in some detail these 84 tests, where every important point necessary for determining the relative values of the standard and Bethlehem beams by direct measurement was actually passed and not recorded in each or any of the 84 tests, the author concludes with a query whether the implied superiority of the standard beams over the Bethlehem, as he assumes to be shown by the data here published regarding these tests, "indicates a corresponding superiority in permanent capacity" (whatever permanent capacity of a beam may mean here); and suggests that this and other equally pertinent questions "are questions to be decided by scientific experiments." He evidently does not rate very highly the efforts of those who conducted the tests that he has recorded in this paper.

In the construction of the new 22d Regiment Armory, at West 168th Street, New York City, the writer specified and used about 1 000 tons of these Bethlehem shapes out of a total of some 2 000 tons in the structure, and including a number of the larger sizes.

Mr. Worthington.

All steel for this work was subjected to rigid inspection at each stage of manufacture, and no difficulty was encountered in getting material that conformed to the specifications, which were practically the same in respect to quality of material as the standard specification of the American Railway Engineering Association for bridge steel.

Throughout the work the writer required that all beams of 12-in. depth and greater, not provided with connection angles framing into other beams, should have end stiffeners, set out on fillers to clear the fillets of flanges, and milled to bear against the flange. With this provision, he found that these beams had ample web thickness for all uses in connection with this work.

For one of the larger beams the writer made a load test which is of some interest in connection with this paper. The gallery on each side of the building is carried by a series of single 30-in., 120-lb., Bethlehem beams, spanning 41 ft. between heavy truss supports, and connected thereto by the standard connection for such beams, namely, two 4 by 4 by $\frac{3}{8}$-in. angles 2 ft. 1 in. long. Alternating with the 41-ft. span are 11-ft. spans, where the corresponding beams were 10 in. deep, so that there was practically no continuity to affect the action of the 30-in. beams under load.

The floor load is transmitted to the 30-in. beams by tiers of small beams at intervals of 2 ft. 9 in., bearing at one end on a wall, the inner face of which was 8 ft. $2\frac{1}{2}$ in. from the center of the 30-in. beam, and cantilevering 7 ft. on the opposite side of the 30-in. beam, thus practically balancing the floor load over the 30-in. beam, and presenting a good opportunity for placing a test load.

There were fourteen such spans, and one was chosen at random for loading with the full live load of 90 lb. per sq. ft. of floor.

The sub-contractor for the concrete floor construction, Mr. Arthur Greenfield, in courtesy to the architects of the building, placed, under the writer's direction, 605 bags of cement on this panel, 121 bags on each of the five seat tiers, and distributed uniformly over the entire length of the span. There was no contact between the adjacent tier loads, and, as stated above, this entire load was practically balanced over the 30-in. beam, resulting in a positive loading of about 52 000 lb. on the beam, uniformly distributed over the entire span.

The only influences tending to modify this distribution of loading on the beam were the following:

(*a*) The floor-joists, 6 in. deep, were built into the wall not more than 8 in., excepting for a space of 16 ft. at the middle of the span, where even this state of restraint was interrupted by large window openings.

(*b*) The ends of the 30-in. beams framed up against 10-in. beams at each end, which tended to produce a state of continuity to a very slight and practically negligible extent.

Mr. Worthington. (c) The rough cinder concrete risers, four in all, each about 3 in. thick and 1 ft. 2 in. high, were in place, with the rough floor slab, and possibly relieved the 30-in. beam of a very small part of its calculated load.

The deflection of the 30-in. beam under this loading was measured at the center of the span and at the quarter points, with reference to the drill floor some 10 ft. below. All steel work at this time was completely riveted up, and the concrete floor slabs of the drill floor had been completed for some time, so that the conditions were very favorable for obtaining accurate results.

The measured deflections of this 30-in. beam were $\frac{1}{4}$ in. and $\frac{3}{8}$ in. at the quarter point and center, respectively; and measurements taken after the load was removed showed a complete recovery at each of these points. All measurements were made by the writer, and were checked up simultaneously by two others.

The calculated deflection of the 30-in. beam under a total uniformly distributed load of 52 000 lb., and a span of 41 ft., assuming a coefficient of elasticity of 30 000 000, is 0.509 in., which compares with the foregoing $\frac{3}{8}$-in. center deflection found by direct measurement. The reason for this difference of about $\frac{1}{8}$ in. is not known to the writer, but it will be seen to lie on the safe side, or that of greater rigidity.

The loading of this 30-in. beam before applying the test load was 583 lb. per lin. ft., giving a calculated extreme fiber stress of 4 200 lb. per sq. in., and the test load of 52 000 lb. amounts to 1 260 lb. per lin. ft., giving a calculated extreme fiber stress of 9 100 lb. per sq. in., so that the total extreme fiber stress under the test load amounted to 13 300 lb. per sq. in.

The one interesting conclusion of importance to be derived from this test is the fact that this beam was perfectly elastic between fiber stresses of 4 200 and 13 300 lb. per sq. in. While it was a single test, the conditions under which the test was made give it greater value than most laboratory tests. It was a beam in actual duty, with standard end connections, with standard and usual joist connections, with top flange braced laterally by a cinder concrete floor slab, all as in usual practice for such beams. The application of the test load was positive and certain, and the measurements of deflections were so simple and easy as almost to preclude possibility of mistake, while the massiveness of the supports practically eliminated all possibility of including in the deflection measurements any other elements of distortion than the bending strains in the beam.

It might be well to add that this beam had holes cut in its web, at intervals of 2 ft. 9 in., about 4 in. wide and about 6 in. high, approximately at the center or neutral axis of the beam. The floorbeams extended through these holes, and there was no reinforcement of the web in any way.

The writer is of the opinion that the Bethlehem beams are a real improvement over the standard sections, theoretically and practically, and all tests that have come to his notice have tended to confirm that opinion. They conform to theory, and are well within the requirements of the best practice in proportions, while the process of their manufacture itself should, by the greater amount of working to which they are subjected, produce a material of more uniform quality than the standard shapes. Mr. Worthington.

When his client can have the same carrying capacity by the use of some 10% less material, there seems to be no good reason for the Engineer to deny him this privilege, except on good grounds, and the writer has yet to discover those grounds.

EDWARD GODFREY, M. AM. SOC. C. E. (by letter).—This very able paper demonstrates clearly the inapplicability of the common theory of flexure to very short, deep beams and girders. Careful designers will generally reinforce such girders, under heavy loading, by adding web plates. These, being usually neglected in the moment of resistance, are a factor tending to offset the deficiency of the flanges. Short beams under light loads are usually made of standard shapes, and have an excess of strength. Mr. Godfrey.

There is a danger, in designing all short, deep beams, especially built beams, that the designer will overlook the importance of proper provision for transmitting flange stress into the web, or, in other words, taking care of horizontal shear. This is true, not only of steel beams, but also of reinforced concrete and even of wood. Short, deep beams of wood are apt to shear horizontally, and should not be used to their full capacity for bending. In reinforced concrete, many designs of short, deep beams have inadequate anchorage or grip. In the tail of a bascule girder the writer has been compelled to extend the cover-plates beyond their apparent limit of usefulness in order to get in enough rivets for their maximum stress; he has had to correct the details of many designs of floor-beams in railroad bridges because there were not enough rivets in the flanges between the girder and the web splice or edge of the gusset-plate. It is very clear that short, deep beams require a care in detailing which is not so manifest in longer beams.

Mr. Prichard refers to the tests made by Professor Marburg on standard and new types of beams, and cites tests which purport to show that the latter, in some cases, show a very low elastic limit. The writer has called attention, a number of times, to the fact that the results of these tests are practically valueless, because the beams had no lateral support whatever. It is true that Mr. Prichard makes the statement that the beams "had no lateral support even at the ends, a severe combination of conditions, rarely encountered, which cannot be regarded as good practice," but this does not state the case ade-

Mr. Godfrey. quately. The error in these tests does not merely consist in having the beams conditioned so that they violate the rules of good practice, for elementary and useful principles can often be discovered by such means; but the error does consist in a totally false interpretation of the tests, a misplaced application of theory, the attributing of results to flexure in beams when flexure contributed but a fraction of those results.

Professor Marburg's beams, those with thin webs and wide flanges which seemed to have such low elastic limits as 10 800 lb. per sq. in., did not fail in flexure; and yet the theory of flexure is applied to measure the elastic limit. It is no more correct to gauge the elastic limit in flexure by the measured deflection of these beams than it would be if the supports were yielding, and this yielding were interpreted to be a giving way of the beams.

The high beams failed by a leaning of the web in opposite directions at each end, that is, by a large twist in the web. This leaning of the web will account for the deflection and apparent yielding of the beam in flexure. In reality, the beam might be perfectly rigid and unyielding in the plane of its web and yet show a large deflection when the web is twisted. The reason for this is very simple to understand. The middle points of the web at the ends determine the middle line of the beam; if this describes an arc over the support, it will drop by an amount equal to the versed sine of that arc.

The higher the web and the greater the area of the flange, the lower will be the apparent elastic limit on this false basis; for, in the high beam, the web will more readily twist, and when the section modulus is large the apparent elastic limit is small (because of the low load required to twist—not deflect—the beam in bending). Thus, between two beams, one stocky in cross-section and the other high, like a plate girder, the first will withstand to better advantage the leaning or twisting of its web, whereas, the other will be easy to tilt over. The latter may have a section modulus several times as great as the former, but both (as shown by deflections) may have the same apparent elastic limit if tested with no lateral support to their compression flanges

Webs of beams are not intended to impart lateral rigidity to the flanges, and it is manifestly an unfair test to compare results between thick and thin webbed beams where the flanges are not stayed laterally. It is in no sense a measure, either of the strength of the metal of the beams, or of the value of the rolled section.

The writer has described* some comparative tests on small beams made of tin plate. In these the mere addition of end stiffeners to prevent the web from leaning added 129% to the ultimate carrying capacity. In another set of tests made recently, also on beams constructed of tin plate, wooden blocks were used as stiffeners on two identical model beams. In one the blocks fitted against the flanges, in the

* *Engineering News*, January 6th, 1910.

other they were chamfered off to allow the web to tilt. The latter showed an apparent elastic limit of less than half that of the former, as indicated by deflections. In fact, the beam with the square blocks was elastic up to the point of failure, and failed by the wedge crushing into the flange and distorting the web. The web of the other beam leaned over about 40° at each end, and the flanges remained quite straight in their own plane. Mr. Godfrey.

In all these tests made by the writer the compression flanges were unstayed, as were those made by Professor Marburg, excepting for the stiffeners in the web used by the writer. They were not made, nor interpreted, to show anything regarding the theory of flexure, however, for in the unstiffened beams flexure contributed but a small part of the deflection.

It is a matter of surprise to the writer that Professor Marburg's tests should be referred to as demonstrating anything regarding the theory of flexure, for, as the writer stated in the reference already cited, "This same kind of test could be made on a plate girder and its elastic limit could be found as low as desired, if such ill-considered tests be taken at their face value." It would be well if some of the literature of engineering could be recalled, or labeled, "Use with caution."

Mr. Vilar y Boy.

S. Vilar y Boy, Assoc. M. Am. Soc. C. E. (by letter).—This interesting paper shows exactitude in calculations based on accurate experiences as well as on the author's especial mathematical knowledge, but, as for the capacity of the beams computed from the refined moment equation, allowing for the deformation of cross-sections, it is possible that the decreases shown in Table 1 are balanced by some unexpected increases due to the actual nature and value of the stresses developed in the material under transverse loads.

It is not the results of personal experiments that have led the writer to this opinion, but simply the values of the center breaking loads for different materials, as given in the handbooks. For instance, compare the center breaking load for a 1-in. square steel beam having a span of 12 in., which Trautwine shows as 5 000 lb., with the breaking load computed according to the ordinary theory, assuming that the stresses counteracting the breaking load are simply compression and tension, above and below the neutral axis, respectively.

$$\text{Bending moment} = \frac{W}{2} \times 6;$$

$$\text{Resisting moment} = S\,\frac{b\,d^2}{6};$$

$$b = 1 \text{ in.};$$

$$d = 1 \text{ in.}$$

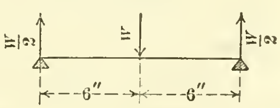

Fig. 11.

$$\text{Therefore, } \frac{W}{2} \times 6 = \frac{S}{6};\ S = 18\,W;\ \text{and } W = \frac{S}{18}.$$

Mr. Vilar y Boy.

Assuming the ultimate strength of steel at 65 000 lb., the center breaking load, W, should be 3 611 lb., as compared with 5 000 lb. obtained by actual tests. Similar results could be computed for other metals and for wood, showing that it is a general law.

Therefore, the ordinary theory, disregarding the deformation of cross-sections, gives results which are too high when compared with those obtained by taking such deformation into account; but, at the same time, the ordinary assumption that flexure is only counteracted by compression and tension in the well-known conditions, gives results which are too small when compared with actual tests for center breaking loads—say, 72%, in the foregoing example.

Therefore the computations of capacities of beams by the ordinary theory, but referred to a certain modulus or coefficient of stiffness to be determined by actual tests, do not appear to be altogether unreasonable; and, in such a case, it would be found that the practical working stresses accepted to-day, referred to such a modulus of stiffness, correspond to a safety coefficient higher than the one now referred to tensile and compressive ultimate strengths.

Mr. Dunham.

H. F. Dunham, M. Am. Soc. C. E.—Common rails of modern heavy type are in many respects similar to I-beams. After they are rolled, they are straightened under heavy presses. How does this process affect the internal stresses in the rail? The latter must be subject to the same law which it has been said operates to improve the stress conditions of an I-beam. In its effect on the steel, is the straightening at the mill very different from the bending of the rail under a locomotive when the ties are not well supported?

It would be encouraging to believe that either process must prove beneficial, but what real evidence is there?

Recorded observations as to the relative frequency of breakage in light and heavy rails, under proportional loads and on the same or similar roadbeds, would be more valuable than theories about the removal of strains and the improvement of the beam or rail by subjecting it to stresses that produce deformation. How do those stresses compare for intensity in light and heavy I-beams, or in rails of widely varying weights? Or, to put the question in another way, do the theories and records show that I-beams and rails of light pattern fail more frequently than do heavy shapes under proportional loads?

Mr. Stern.

Eugene W. Stern, M. Am. Soc. C. E.—This admirable paper is very interesting, and the author discusses a number of points to which the speaker has given attention in his practice. The illustration of the diminution in strength of a channel due to unsymmetrical loading has been amply verified in the speaker's experience. An occurrence of many years ago has never been forgotten, as the results were so surprising: A line of shafting was supported on some 12-in. channels in which the factor of safety, if calculated by ordinary methods, was fully 20. Under the weight of the shafting and the pull of the belts, these channels actually bent over and collapsed. As very often happens when material leaves the mills, there was a slight initial bend in the webs of the channels, which had not been straightened in the fabricating shops, and, besides, their ends simply rested on girders without any provision for bracing them in an upright position. The fault was rectified by taking out the channels, straightening them, tying the ends together, and bracing them so that they could not bend sideways, and no further difficulty was ever experienced. Mr. Stern.

With reference to internal stresses in a beam due to cooling after being rolled, Mr. Prichard's remarks are confirmed by the speaker's experiences. Some years ago he had occasion to shear some 8-in. beams, 10 ft. long, down the middle of the web in order to make 4-in. T's out of them, and it was found that each half of the beam curved outward about 2½ in. This happened, not with one beam alone, but almost uniformly with about twenty.

Referring to the comparison between the strength of beams of the Standard and Bethlehem sections, given in Mr. Prichard's tables, the speaker would very much like to know under what conditions these tests were made and by whom, because he has been using a great many Bethlehem sections on account of their supposed economy over Standard sections. If they are not as strong as the Standard sections, he, of course, would like to know it. The only tests of which he has seen a record were those made by Professor Marburg, which are alluded to by Mr. Prichard. To the speaker's mind, these tests were of no value for the reason that they were made under conditions which, in all his practice, he has never seen in building or bridge construction. No attempt whatever was made to prevent the webs from buckling, nor to brace the ends of the beams laterally. These Marburg tests on the full-sized sections convinced the speaker of only one thing, namely, that they are of no service to the practicing engineer. Therefore, it would be of great interest to know just how these later tests, mentioned by the author, were made and by whom. It is hoped that Mr. Prichard will make a detailed statement of this matter.

Tests of full-sized sections for comparing the relative strengths of different designs will be of value only if exactly the same conditions are adopted for the testing of every piece, and likewise if the method of testing is devised in such a way as to impose conditions of loading which might commonly be obtained in practice. If the beams in one particular set were not straightened and trued up as carefully as those in another set, before testing, the latter, of course, would be expected to give higher values. The case of the channels, cited by the speaker, Mr. Stern.

in which a little straightening of the webs and the use of a better type of end connections very materially strengthened them, emphasizes this point.

Another important point which Mr. Prichard has brought out is the necessity of using stiffeners in the webs of beams. Many beams have a slight curvature in the web when they come from the mill; likewise, the flanges are not exactly parallel. If, therefore, heavy concentrations are applied to the top flanges of beams, such as, for instance, in the grillage beams under columns, the crippling of the webs must be carefully guarded against, and, in such cases, it is the speaker's invariable practice to use stiffeners ground to fit tightly between the flanges, whether or not the calculations require them.

Referring to the theory of flexure as applicable to short beams, will the author be so kind as to state, according to his theory, what the strength would be of, say, a 20-in., 100-lb. beam, 6 ft. long, as compared with that given in the handbooks?

Mr. Belzner.

THEODORE BELZNER, ASSOC. AM. SOC. C. E. (by letter).—Mr. Prichard states that many experiments have shown that the over-straining of a beam lowers the elastic limit of the metal in it temporarily, and that this elastic limit returns after a rest. If such is the case, it would be very interesting to know when the normal elastic limit has again been reached, and also the length of time necessary for a complete recovery; for example, in the case of I-beams varying from 12 to 20 in. in depth.

Mr. Williams.

J. P. J. WILLIAMS, ASSOC. M. AM. SOC. C. E.—Attention is called to two important points: first, the relative magnitude of the intensities of initial stresses compared with the so-called errors in theory with which the paper is concerned; and second, the actual method of derivation of the values in Table 1 by the difference, z, representing the effect of shear strain.

The first question, concerning the actual magnitude of initial stresses, is one which would require careful experiment for its settlement. The only record of such experiment that the speaker recalls is one given by the late J. B. Johnson, M. Am. Soc. C. E., in describing the tests for such initial stresses in a thick cast-iron cylinder for a large gun, in which intensities of stress as high as from 5 000 to 10 000 lb. per sq. in. were found. The example of a rolled I-beam in which the web had been split or torn apart at the ends by the shrinkage stresses in the flanges certainly suggests the probability of large intensity of shrinkage stress. It is impossible to determine theoretically just what that stress would be, because the most arbitrary assumptions must be made regarding the distribution of the internal stresses. It is well to bear in mind, however, that there must inevitably be present in all rolled sections relatively large and unknown internal stresses, due not only to the unequal cooling in manufacture, but also to the subsequent straightening and shop manipulation. Only the results of experimental tests can be considered as final evidence of the character and importance of such stresses, and the speaker is inclined to agree with the criticism which was made of the paper because of its meager and incomplete report of experiments which were probably of great practical value as exhibiting the actual characteristic test strength of rolled beams. As Professor Marburg obtained such astonishingly low values in his tests of large beams, further test results are greatly to be desired, especially for such practically framed beams as those having the deflections reported by the author, and which were apparently free from the criticism justly applied to Marburg's tests. Mr. Williams.

The second point raised concerns the method of ascertaining the values given in Table 1 as the percentage of error in the results of the common theory of flexure. First, however, the speaker desires to express his appreciation and give his endorsement of the author's objects in presenting this paper, and his attempt to determine the actual resulting limitations which the more complete general theory of flexure imposes on the use of the ordinary formulas. The speaker finds, however, that there can be developed a theoretical treatment of the particular error considered in Section 1, which is perhaps more logical, and certainly gives a more general formula showing exactly what elements affect the problem. The fundamental basis of this treatment is the well-known law of equality between the internal resisting moment at any point and the external bending moment at that point. Whatever assumption may be made regarding the variation of stress intensity or the character of original plane sections after flexure, the resulting total internal resisting moment must remain the same and be equal to the bending moment at that section. As all beam design is based on the computed value of the extreme maximum fiber stress, it seems best to use the actual value of such extreme fiber stress as a measure of the error in the common theory. This is the distinct difference in the methods used: The author finds a new actual value for the resisting moment, M', by what he calls "a refined method"; while the speaker has found a new actual value for the extreme fiber stress intensity by considering shear strain or distortion.

That, under certain conditions, there is an error in the usual common theory, due to the assumption of plane normal sections, is at once evident when the facts of shear distortion or strain are considered, as is indicated by the author in Fig. 3. To make the actual cause of this distortion of the plane normal section more evident, consider Fig. 12, showing the upper half of a vertical section, dx in width. Assuming the neutral axis, N-A, as the fixed line of reference, the presence of shearing stress in the vertical planes induces equal intensities of shearing stress in the horizontal planes, and hence horizontal

Mr. Williams. shear strains or distortions which increase continually in total amount as the point considered approaches the extreme fiber.

Making the same fundamental assumption as is made by the author, that the secondary effect of the error in fiber stresses on the shearing stresses themselves can be neglected, a formula for the difference, *e*, in intensity of extreme fiber stress, as usually computed and as obtained by including shear distortion, is found to have the following form, as will be derived later. (Equation 10.)

$$e = \frac{dS\ E}{dx\ I\ G}\left(\frac{K}{O} - K_e\right)$$

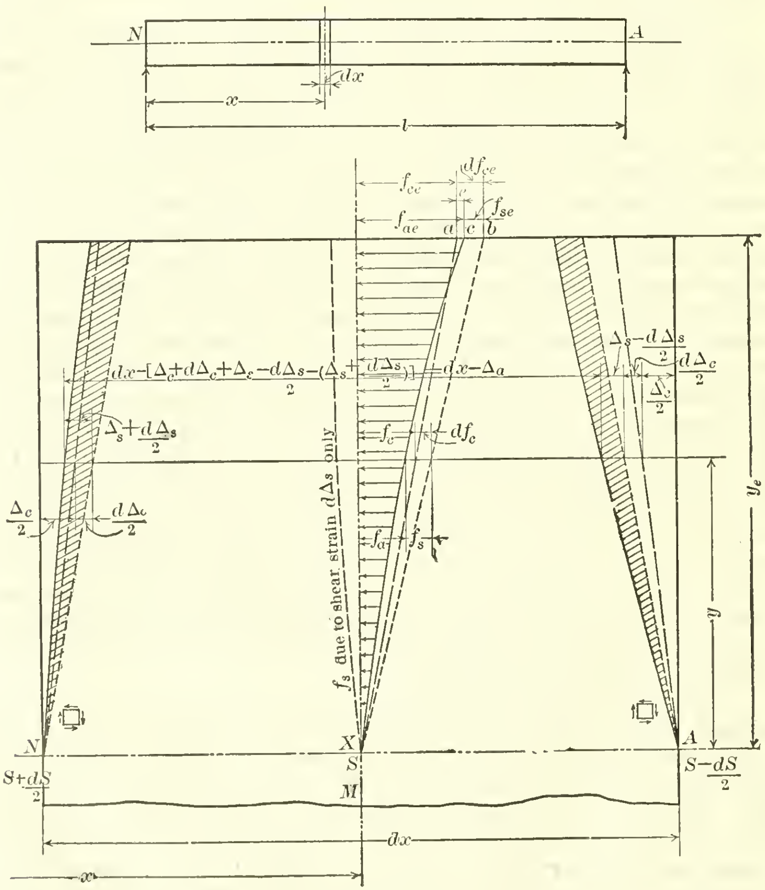

Fig. 12.

In this equation, K and K_e are constants depending simply on the character and dimensions of the normal cross-section, O being the section modulus of that section, and I its moment of inertia. The term, $\frac{dS}{dx}$, represents simply the change in shear per unit length at the point considered, and E and G are the coefficients of elasticity for direct and shearing stresses, respectively. Mr. Williams.

This equation gives results on the side of danger which, for rectangular sections, agree fairly well with those given by the author in Table 1; but, for the girder sections, the percentage of error is found to be considerably less, as will be shown in more detail later. The importance of the factor, $\frac{dS}{dx}$, should be emphasized. It shows that, where the shear is constant, there is no error whatever in the common theory. This is due to the fact that the departure from the plane normal sections assumed in the common theory of flexure is identical for successive sections when the shear is constant. This fundamental assumption of the common theory is never precisely true, as all normal sections are curved by shear distortion. When a beam is loaded with a uniform load, w per in., $\frac{dS}{dx} = w$, and the error, e, is constant throughout. When a concentrated load occurs, the value of $\frac{dS}{dx}$ may be large, and depends directly on the assumed horizontal distance over which such concentration is distributed.

This value of e, therefore, may be large for short beams with large values of w, and also at points of concentrated loading, and it depends directly on how such loads are applied and distributed. Thus the points raised by the author in Section 2 as to the manner of loading are also of importance in relation to the error in fiber stress. The author's emphasis of this point is greatly needed, and the practice of applying large concentrated loads to beams without adequate means of distribution, both vertically and horizontally, at the local points of application, is shown to be dangerous. The actual resulting intensities of stress at such loads are theoretically impossible of determination. It should be noted that the foregoing formula has neglected entirely the local lateral strains and vertical stresses induced by such concentrated application of loads. The speaker has approximated roughly the effect of the lateral strain, and finds that its maximum value is probably very small; but the buckling effect on the web, of concentrated loads applied to the flange, should always be guarded against by stiffeners. Just what would be the limiting value of the concentration that can be carried by a given web, is really, in the speaker's opinion, an empirical problem. The attempt to use the general formulas of the complete

Mr. Williams. theory requires an immediate assumption as to the distribution of the vertical direct stress, and leads to very complicated analysis.

Derivation of Formula for the Error, e.—The following notation is used for the fiber at the distance, y, from the neutral axis, N-A:

Δ_c = Total strain in the length, dx, by the common theory;
$d\Delta_c$ = Change in Δ_c when the shear strain is considered;
Δ_s = Total shear strain due to the shear, S;
$d\Delta_s$ = Change in Δ_s in the distance, dx, due to the change, dS, in the shear, S;
Δ_a = Total actual final strain in the length, dx;
f_c = Intensity of direct fiber stress by the common theory $= \frac{My}{I}$;
f_a = Actual intensity of direct fiber stress, including the effect of shear;
f_s = Intensity of direct fiber stress corresponding to $d\Delta_s$, that is, $f_s = \frac{d\Delta_s}{dx} E$;
s = Intensity of shearing stress $= G\,\phi$;
b_1 = Breadth of normal cross-section;
m = Static moment about N-A, of area outside the horizontal section considered.

Also, for the extreme fiber at the distance, y_e, from the neutral axis, N-A, similarly:

f_{ce} = Intensity of direct fiber stress by the common theory;
f_{ae} = Actual intensity of direct fiber stress, including the effect of shear;
f_{se} = Intensity of direct fiber stress corresponding to $d\Delta_{se}$;
e = Intensity of direct fiber stress representing the error of the common theory, that is, $e = f_{ae} - f_{ce}$.

Also:

A = Area of normal cross-section of beam;
I = Moment of inertia;
O = Section modulus $= \frac{I}{y_e}$;
w = Weight of uniform load per inch;
l = Length, in inches;
ϕ = Rate of shearing strain or distortion $= \frac{s}{G}$;
E = Coefficient of elasticity for direct stress;
G = Coefficient of elasticity for shearing stress.

Considering the total strains shown in Fig. 12 for the end planes at N and A, the hatched areas represent the strain due to horizontal shear, drawn to no scale. Such shear strains are the cause of the distor-

Mr. Williams.

tion of the plane section, and they result in an added direct strain and increase in stress required to keep the value of the internal resisting moment the same as before. The dash lines represent strains and corresponding stress intensities for the common theory. The dotted lines show the increase in the straight-line portion of the resulting strains and stresses, these differential increases being called $d\Delta_c$, df_c, etc. The total actual decrease in length of fiber at the distance, y, from N-A is, as shown by Fig. 12, $\Delta_a = \Delta_c + d\Delta_c + \Delta_s - \frac{d\Delta_s}{2} - \left(\Delta_s + \frac{d\Delta_s}{2}\right)$, which gives $\Delta_a = \Delta_c + d\Delta_c - d\Delta_s$, in which $d\Delta_s$ is the change in shear strain, Δ_s, due to change in shear, dS, in the distance, dx. Therefore, by the definition of E, the actual intensity, f_a, would be $f_a = E\frac{\Delta_a}{dx} = \frac{E}{dx}(\Delta_c + d\Delta_c - d\Delta_s)$; but $\frac{E\,\Delta_c}{dx} = f_c$, $\frac{Ed\Delta_c}{dx} = df_c$, and $\frac{Ed\Delta_s}{dx} = f_s$. Therefore

$$f_a = f_c + df_c - f_s \quad \text{.....................} (1)$$

On the center line in Fig. 12, these intensity of stress lines have been drawn to indicate the probable character of the variation across the section, and the value for f_a in Equation 1 also results directly from the figure as drawn. The necessity for the increase, df_c, in fiber stress intensity, is seen in the shape of the final limiting line of actual stress intensities, f_a, which is concave toward the vertical plane considered, tending to decrease the intensities as compared with the straight-line variation, and hence decrease the resisting moment. As has been stated, the fundamental relations of the beam require this resisting moment to remain constant and equal to the constant external bending moment. Therefore the required increase, df_c, in the assumed straight-line portion of the stress intensities must result.

The values in Equation 1 can now be found. From the foregoing assumed straight-line variation in intensity:

$$df_c = df_{ce} \times \frac{y}{y_e} \quad \text{.....................} (2)$$

From the consideration of the total difference in direct fiber stress on the two sides of a notch, dx in width, according to the common theory, the intensity of shearing stress at y from N-A is $s = \frac{S\,m}{b_1\,I}$ (Equation 5, p. 905 of the paper), and therefore $ds = \frac{dS\,m}{b_1\,I}$. To find $f_s = \frac{d\Delta_s}{dx}E$, the value of $d\Delta_s$ must be found by integrating from 0 to y all the differential horizontal shear strains, $d\phi \times dy$. By the definition of G, the value of $d\phi$ becomes: $d\phi = \frac{ds}{G} = \frac{dS\,m}{b_1\,I\,G}$, and therefore $d\Delta_s = \int_0^y d\phi \times dy = \frac{dS}{I\,G}\int_0^y \frac{m\,dy}{b_1}$.

Mr. Williams.

The value of f_s will then be:

$$f_s = \frac{dS}{dx}\frac{E}{I\ G}\int_0^y \frac{m\ dy}{b_1} \quad \text{.................... (3)}$$

For the extreme fiber stress intensity, f_{se}, let $y = y_e$:

$$f_{se} = \frac{dS}{dx}\frac{E}{I\ G}\int_0^{y_e} \frac{m\ dy}{b_1} = \frac{dS}{dx}\frac{E\ K_e}{I\ G} \quad \text{............ (3a)}$$

$$\text{in which } K_e = \int_0^{y_e} \frac{m\ dy}{b_1} \quad \text{.................. (4)}$$

Equation 1, for the actual intensity at y from N-A, then becomes:

$$f_a = f_c + df_{ce}\frac{y}{y_e} - \frac{dS}{dx}\frac{E}{I\ G}\int_0^y \frac{m\ dy}{b_1} \quad \text{.............. (5)}$$

Now, the fundamental relation which must always hold, that is, the equality of the internal resisting moment and the external bending moment, M, gives the following equation:

$$M = \int^A f_a\ dA\ y =$$

$$\int^A f_c\ dA\ y + \frac{df_{ce}}{y_e}\int^A y^2\ dA - \frac{dS}{dx}\frac{E}{I\ G}\int^A \left(\int_0^y \frac{m\ dy}{b_1}\right) dA\ y \ldots (6)$$

As in the common theory of flexure, $f_c = f_{ce}\dfrac{y}{y_e}$, therefore the first term becomes:

$$\int^A f_c\ dA\ \ y = \frac{f_{ce}}{y_e}\int^A y^2\ dA = \frac{f_{ce}\ I}{y_e} = M, \text{ by the definition of } f_{ce}.$$

The second term becomes $\dfrac{df_{ce}\ I}{y_e}$, and if, for simplicity in notation, we let

$$K = \int^A \left(\int_0^y \frac{m\ dy}{b_1}\right) y\ dA \quad \text{................ (7)}$$

Equation 6 becomes:

$$M = M + \frac{df_{ce}\ I}{y_e} - \frac{dS}{dx}\frac{E\ K}{I\ G}.$$

This equation can at once be solved for the unknown, df_{ce}, which gives the value of the differential change in extreme fiber stress, as follows:

$$df_{ce} = \frac{dS}{dx}\frac{y_e\ E\ K}{I^2\ G} \quad \text{.................... (8)}$$

The value of df_{ce} can now be substituted in Equation 5, and there results:

$$f_a = f_c + \frac{dS}{dx}\frac{E}{I\ G}\left[\frac{y\ K}{I} - \int_0^y \frac{m\ dy}{b_1}\right] \quad \text{............ (9)}$$

To obtain the value of the error in the intensity of extreme fiber stress, we have finally, from Fig. 12: Mr. Williams.

$$e = f_{ae} - f_{ce} = df_{ce} - f_{se}.$$

Substituting the values of df_{ce} and f_{se} given by Equations 8 and 3a:

$$e = \frac{dS\, E}{dx\, I\, G}\left[\frac{y_e K}{I} - K_e\right].$$

Or, since $\dfrac{I}{y_e} = O$,

$$e = \frac{dS\ E}{dx\ I\ G}\left[\frac{K}{O} - K_e\right] \ldots\ldots\ldots\ldots\ldots\ldots \quad (10)$$

The percentage of error would depend on f_{ce}, and would be:

$$\frac{100\ e}{f_{ce}} = \frac{100\ e\ O}{M} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (11)$$

Note that when e is positive the actual maximum fiber stress intensity, f_{ac}, is greater than f_{ce}; therefore, the value obtained by the common theory is in error on the side of danger.

Application to Rectangular Beam.—Consider a beam of rectangular cross-section, b in width and d in depth, for which $I = \dfrac{bd^3}{12}$, $O = \dfrac{bd^2}{6}$,

$$b_1 = b = \text{constant},\ m = \int_y^{y_e} y\, dA = b\int_y^{\frac{d}{2}} y\ dy = \frac{b}{2}\left(\frac{d^2}{4} - y^2\right).$$

To find the values of K and K_e in Equations 7 and 4, we have:

$$\int_0^y \frac{m\ dy}{b_1} = \frac{b}{2\,b}\int_0^y \left(\frac{d^2}{4} - y^2\right) dy = \frac{y}{24}\ (3\ d^2 - 4\ y^2) \ldots\ldots \quad (12)$$

$$\text{and } K_e = \int_0^{y_e} \frac{m\ dy}{b_1} = \frac{d^3}{24} \ldots\ldots\ldots\ldots\ldots \quad (12a)$$

by making $y = y_e = \dfrac{d}{2}$ in Equation 12.

Then, by Equation 7,

$$K = \int^A \frac{y^2}{24}(3\,d^2 - 4\,y^2)\,dA = \frac{2\,b}{24}\int_0^{\frac{d}{2}} (3\ y^2\ d^2 - 4\ y^4)\ dy,$$

$$\text{or } K = \frac{b}{12}\left[3\ d^2\frac{y^3}{3} - \frac{4\ y^5}{5}\right]_0^{\frac{d}{2}} = \frac{bd^5}{120} \ldots\ldots\ldots\ldots \quad (13)$$

$$\text{By Equation 10, } e = \frac{dS\ E}{dx\dfrac{bd^3}{12}\ G}\left[\frac{\dfrac{bd^5}{120}}{\dfrac{bd^2}{6}} - \frac{d^3}{24}\right] = \frac{dS\ E}{10\ b\ dx\ G} \ldots\ldots \quad (14)$$

$$\text{By Equation 11, the percentage, } e = \frac{10\ dS\ \ E\ bd^2}{6\ b\ dx\ G\ M} = \frac{1.67\, dS\ Ed^2}{dx\ G\ M}. \quad (15)$$

Mr. Williams.

Equations 14 and 15 indicate very clearly just how the error, e, and the relative percentage, e, will vary. Thus the narrower the beam, the greater is the absolute value of the error; also, this absolute value increases with an increased ratio, $\frac{E}{G}$, and of course with the increase in shear change, $\frac{dS}{dx}$, as already discussed. The percentage of error, of course, will vary inversely with M, and therefore, for usual design, using the maximum bending moment, this percentage is least.

In order to compare the results of this analysis with the values given by the author in Table 1 for rectangular sections, let Poisson's ratio, r, equal $\frac{1}{3}$, as taken by Mr. Prichard for steel. Then, by the well-known theoretical relation between E and G, we have, $\frac{E}{G} = 2\ (1 + r) = 2.67$.

The author also assumes a uniform load, w, and a fully loaded beam. Therefore, at any point, x, from the end, $M = \frac{w}{2}\, x\ (l - x)$; also, $\frac{dS}{dx} = w$. Equation 15 then gives:

$$\text{the percentage, } e = \frac{1.67\ w\ (2.67\ d^2)}{\dfrac{w\ x\ (l - x)}{2}} = 8.88\ \frac{d^2}{x\ (l - x)} \quad \ldots. \ (15a)$$

The least percentage of error will be obtained at the center, where M is a maximum, and $x = \frac{l}{2}$, giving

$$\text{the percentage, } e = 35.5\ \left(\frac{d}{l}\right)^2 \ldots\ldots\ldots\ldots\ (15b)$$

When $\frac{d}{l} = 1$, the percentage, $e = 35.5\%$ *versus* 37% in Table 1.

When $\frac{d}{l} = \frac{1}{10}$, the percentage, $e = 0.35\%$ *versus* 0.51% in Table 1.

At other points along the beam, the results are larger; for instance, at the quarter point, where $x = \frac{l}{4}$, the percentage, $e = 47.3 \left(\frac{d}{l}\right)^2$ giving 47.3% when $\frac{d}{l} = 1$ and 0.47% when $\frac{d}{l} = \frac{1}{10}$. It is seen that the percentage, e, varies with the square of $\frac{d}{l}$, and the vital influence of this ratio of depth to length on the resulting error should be noted.

Influence on Deflection.—The common theory of flexure results in deflection equations which are separated into two classes: bending deflection equations and shear deflection equations. The former give

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a deflection constant for all fibers of the beam, regardless of their position with respect to the neutral axis; and the latter give the deflection of the neutral axis, usually, the deflection of the extreme fibers due to shear being zero, since shear at that point is zero. It would thus appear from these formulas that the neutral axis would deflect more than the outside layers, tending to induce vertical stress intensity, and separate the longitudinal layers. The effect of lateral strain, in changing the lateral dimensions, and in causing a slight downward movement of the neutral axis with respect to the outside fibers, can be shown to be about one-tenth of the shear deflection at the center of a simple rectangular beam uniformly loaded.

It is interesting to find that if this lateral strain be neglected, the error in the common theory, as above developed, will exactly account for the apparent movement of the neutral axis downward by the amount of the shear deflection of that axis. This forms a good mathematical check of the foregoing theory and formulas, and also makes clear the fact that there can really be no vertical stress developed between the longitudinal layers, as would be indicated by the common theory. Consider a rectangular beam uniformly loaded:

Let D_s = the center deflection of the neutral axis due to shear;
D_e = the center deflection of the extreme fiber due to bending;
D_o = the center deflection of the neutral axis due to bending.

The deflections, D_e and D_o, would be the same if the common theory, based on plane sections after flexure, were applied. The value of D_s—remembering that for the neutral axis $s = 1.5 \frac{S}{bd}$, and $S = w\left(\frac{l}{2} - x\right)$ — would be

$$D_s = \int_0^{\frac{l}{2}} \phi \, dx = \int_0^{\frac{l}{2}} \frac{s}{G} \, dx = \frac{1.5 \, w}{bd \, G} \int_0^{\frac{l}{2}} \left(\frac{l}{2} - x\right) dx = \frac{0.187 \, wl^2}{bd \, G} \qquad (16)$$

Now, to find values for D_e and D_o, the fundamental relations indicated in Fig. 13 can be applied. This figure shows that any small actual angular change in the original position of a normal plane will deflect the corresponding fiber a distance, $dD = x \tan. \alpha_a = x \, \alpha_a$, as these angles are so small that they can be considered equal to their tangents.

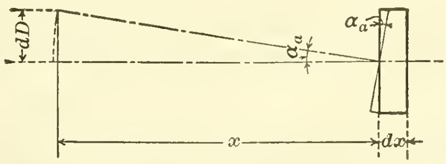

FIG. 13.

The value of α_a for the case considered can be found by using

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the values of actual strains, Δ_a, corresponding to the actual fiber stress intensities, f_a, at the distance, y, from the neutral axis. The rate of strain being $\frac{f_a}{E}$, the total strain in the length, dx, would be $\Delta_a = \frac{f_a dx}{E}$. The value of f_a can be found from Equation 9 and Equations 12 and 13 for rectangular sections:

$$f_a = f_c + \frac{dS}{dx}\frac{E}{I\,G}\left[\frac{y\,b\,d^5}{120\,I} - \frac{y}{24}(3\,d^2 - 4\,y^2)\right] \quad \ldots\ldots.(17)$$

Now, considering the case of a beam uniformly loaded,

$$M = \frac{w\,x}{2}(l - x), \text{ and}$$

$f_c = \frac{M\,y}{I} = \frac{w\,x\,(l-x)\,y}{2\,I}$; also $\frac{dS}{dx} = w$. Substitute in Equation 17:

$$f_a = \frac{w\,y}{I}\left[\frac{x\,(l-x)}{2} + \frac{E}{G}\left(\frac{b\,d^5}{120\,I} - \frac{3\,d^2 - 4\,y^2}{24}\right)\right] \ldots.(17a)$$

Then the value of actual strain becomes,

$$\Delta_a = \frac{f_a\,dx}{E} = \frac{w\ dx\ y}{I}\left[\frac{x\,(l-x)}{2\,E} + \frac{b\,d^5}{120\,\frac{b\,d^3\,G}{12}} - \frac{3\,d^2 - 4\,y^2}{24\,G}\right]$$

$$= \frac{w\,dx\,y}{I}\left[\frac{x\,(l-x)}{2\,E} - \frac{d^2}{40\,G} + \frac{y^2}{6\,G}\right] \ldots\ldots\ldots\ldots(18)$$

The value of α_a at any distance, y, from the neutral axis is the same as its tangent, as the angle is small, and this tangent is $\frac{d\Delta_a}{dy}$. Therefore:

$$\alpha_a = \frac{d\,\Delta_a}{dy} = \frac{w\,dx}{I}\left[\frac{x\,(l-x)}{2\,E} - \frac{d^2}{40\,G} + \frac{y^2}{2\,G}\right] \ldots\ldots (19)$$

This equation shows that α_a increases as y increases. At the extreme fiber it would be found by letting $y = \frac{d}{2}$ and at the neutral axis $y = 0$. If α_{ae} is the value of this angle at the extreme fiber, and α_{ao} is its value at the neutral axis, the difference between these angles would be, by substituting for y in Equation 19 and subtracting:

$$\alpha_{ae} - \alpha_{ao} = \frac{w\,dx}{I} \times \frac{d^2}{8\,G} \ldots\ldots\ldots\ldots\ldots\ldots (20)$$

It is this difference which causes the final difference in deflection, $D_e - D_o$, which is desired, and can now be found by integrating the value of each differential, $d\,(D_e - D_o)$, for every section between the end and the center. Remembering (as shown by the discussion on Fig. 13) that $d\,(D_e - D_o) = x\,(\alpha_{ae} - \alpha_{ao})$, there results, by using Equation 20 and letting $I = \frac{b\,d^3}{12}$: Mr. Williams.

$$D_e - D_o = \int_0^{\frac{l}{2}} \frac{x\,w\;dx\,d^2}{8\,I\,G} = \frac{1.5\,w}{bd\;G}\left[\frac{x^2}{2}\right]_0^{\frac{l}{2}} = \frac{0.187\,w\,l^2}{b\,d\;G} \ldots (21)$$

This difference in deflection is thus seen to be exactly the same as the shear deflection of the neutral axis, D_s, of Equation 16. Fig. 14 makes it clear that this probably results in the same deflection, D_e, for all layers of the beam, as Equations 21 and 16 give

$D_e = D_s + D_o$, as indicated on the figure.

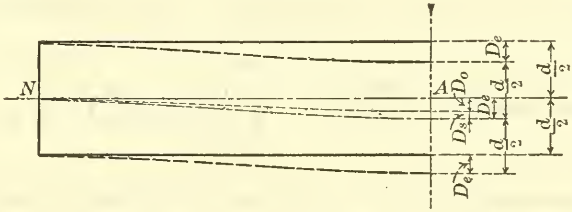

FIG. 14.

This suggests the fact that the ordinary formulas for deflection of beams by the common theory give results which are too small by the value $e_D = D_e - D_c$, if D_c is equal to the deflection at the center by the common theory, due to bending, the value of shear deflection usually being neglected, since it is zero for the lower surface of the beam. It will be interesting to obtain a value for this error in deflection, e_D, for the case of the rectangular beam here considered. As the deflection is being found to a point where the tangent to the neutral axis is horizontal (at the center), the deflection, D_e, is most easily obtained directly by the relations similar to those already used, that is, $dD_e = x\,\alpha_{ae}$, and, therefore, $D_e = \int_0^{\frac{l}{2}} x\;\alpha_{ae}$. Now, the value of α_{ae} results from Equation 19, with $y = y_e = \frac{d}{2}$, and hence there results:

$$D_e = \frac{w}{I}\int_0^{\frac{l}{2}} \left(\frac{x^2\,(l - x)\,dx}{2\;E} + \frac{d^2}{10\;G}\;x\,dx\right) \ldots\ldots\ldots (22)$$

It will be recalled that the first term in this integral is the result of the bending stress, f_c, only, and should check exactly the common theory value, $D_c = \frac{5}{384} \times \frac{w\,l^4}{E\,I}$, as given by all textbooks for this case. Integrating this first term for a check, there is found:

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$$\frac{w}{2\,E\,I}\int_0^{\frac{l}{2}} (l - x)\, x^2\, dx = \frac{w}{2\,E\,I}\left[\frac{l\,x^3}{3} - \frac{x^4}{4}\right]_0^{\frac{l}{2}} = \frac{w\,l^4}{2\,E\,I}\left[\frac{1}{24} - \frac{1}{64}\right]$$

$$= \frac{w\,l^4}{2\,E\,I}\left(\frac{8-3}{192}\right) = \frac{5}{384}\,\frac{w\,l^4}{E\,I} = D_c \ldots\ldots\ldots\ldots (23)$$

The second term in Equation 22, therefore, gives the error, e_D, required, as Equations 22 and 23 give:

$$D_e = D_c + \frac{w\,d^2}{10\,G\,I}\int_0^{\frac{l}{2}} x\,dx, \text{ and, therefore,}$$

$$e_D = D_e - D_c = \frac{w\,d^2}{10\,G\,I}\left[\frac{x^2}{2}\right]_0^{\frac{l}{2}} = \frac{w\,d^2\,l^2}{80\,G\,I} \ldots\ldots\ldots (24)$$

The percentage of error in the deflection, D_c, as usually found, would then be:

$$\text{percentage, } e_D = \frac{100\,e_D}{D_c} = \frac{\dfrac{w\,d^2\,l^2}{80\,G\,I}}{\dfrac{5}{384}\,\dfrac{w\,l^4}{E\,I}} = 96\,\frac{E}{G}\left(\frac{d}{l}\right)^2 \ldots\ldots (25)$$

This percentage of error in deflection is thus seen to vary directly with the ratio of E to G and directly as the square of the ratio of depth to length for a rectangular section. If $\frac{E}{G} = 2.67$ for steel, and $\frac{d}{l} = \frac{1}{10}$, as in usual design limits, there is found:

$$\text{the percentage, } e_D = \frac{96 \times 2.67}{100} = 2.56\% \ldots\ldots\ldots (25a)$$

The general percentage, $e_D = 256\left(\frac{d}{l}\right)^2$ for steel rectangles, showing a large percentage of error as the ratio, $\frac{d}{l}$, increases. Just what the error for rolled steel sections would be, in comparison with rectangular sections, cannot be stated, and, as will be seen later, the analysis for such sections is rather complicated.

It should be noted that the foregoing analysis has been based on the usual common theory values of shearing stress intensity. As was well emphasized by the author in Section 3 of the paper, this shear intensity is really not distributed uniformly across the horizontal plane section considered, as is assumed in the common theory. Neither is the actual value of such shear intensity the same as given by the common theory, as the secondary effect of error in direct fiber stresses would modify the shear values. It would be possible to make successive approximations to the true values of both shear and direct stress formulas, but, as mathematical analysis is already too prominent in this discussion, the speaker will not at this time attempt to do so. Mr. Williams.

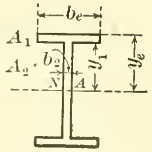

FIG. 15.

Application to Rolled Sections.—The values of K_e and K required in Equation 10 to find the error, e, for rolled sections, can best be found by considering an equivalent section, as shown and dimensioned in Fig. 15.

For the outside rectangle, A_1, the value of the static moment, m_1, would be:

$$m_1 = b_e \int_{y_1}^{y_e} y\,dy = \frac{b_e}{2}(y^2_e - y^2_1) \ldots\ldots\ldots\ldots (26)$$

To find the value of $K_e = \int_0^{y_e} \frac{m\,dy}{b_1}$ from Equation 4: Consider the area, A_1, first, $b_1 = b_e$ and $m = \int_y^{y_e} y\,dA = b_e \int_y^{y_e} y\,dy$; whence,

$m = \frac{b_e}{2}(y_e^2 - y^2)$; therefore

$$\int_{y_1}^{y_e} \frac{m\,dy}{b_1} = \frac{1}{2}\int_{y_1}^{y_e} (y_e^2 - y^2)\,dy = \frac{y_e^2}{2}(y_e - y_1) - \frac{y_e^3 - y_1^3}{6} \ldots (27)$$

Similarly, for the area, A_2, $b_1 = b_2$ and $m = m_1 + b_2 \int_y^{y_1} y\,dy$

$$= m_1 + \frac{b_2}{2}(y_1^2 - y^2);$$

therefore

$$\int_0^{y_1} \frac{m\,dy}{b_1} = \int_0^{y_1} \left(\frac{m_1}{b_1} + \frac{y_1^2 - y^2}{2}\right) dy = y_1\left(\frac{m_1}{b_2} + \frac{y_1^2}{2}\right) - \frac{y_1^3}{6}$$

$$= K_1 - \frac{y_1^3}{6} \ldots\ldots\ldots\ldots\ldots\ldots (28)$$

$$\text{If } K_1 = y_1\left(\frac{m_1}{b_2} + \frac{y_1^2}{2}\right) \ldots\ldots\ldots\ldots\ldots\ldots (29)$$

adding Equations 27 and 28 will give:

$$K_e = \int_0^{y_e} \frac{m\,dy}{b_1} = K_1 - y_e^2\left(\frac{y_1}{2} - \frac{y_e}{3}\right) \ldots\ldots\ldots (30)$$

To find value of $K = \int^A \left(\int_0^y \frac{m\,dy}{b_1}\right) y\,dA$ from Equation 7:

Mr. Williams. This integration can be made by considering separately the rectangles, A_1, and A_2, as follows:

For the area, A_1: To get $\int_0^y \frac{m\,dy}{b_1}$, the static moment, m, for sections in A_1 is $\frac{b_e}{2}(y^2_e - y^2)$, and for sections in A_2 is $m_1 + \frac{b_2}{2}(y_1^2 - y^2)$. Then, as $b_1 = b_e$ for A_1 and b_2 for A_2 the total integral will be,

$$\int_0^y \frac{m\,dy}{b_1} = \int_0^{y_1} \left(\frac{m_1}{b_2} + \frac{y_1^2 - y^2}{2}\right) dy + \int_{y_1}^y \frac{y^2_e - y^2}{2}\, dy$$

$$= \left(\frac{m_1}{b_2} + \frac{y_1^2}{2}\right) y_1 - \frac{y_1^3}{6} + \frac{y^2_e}{2}(y - y_1) - \frac{y^3 - y_1^3}{6}$$

$$= K_1 - \frac{y^2_e\, y_1}{2} + \frac{y}{2}\left(y^2_e - \frac{y^2}{3}\right).$$

Now, $dA = 2\, b_e\, dy$, to include the symmetrical areas on both sides of the neutral axis, therefore, the value of K for the area, A_1, will be,

$$\int_{y_1}^{y_e}\left(\int_0^y \frac{m\ dy}{b_1}\right) y\, dA = 2\ b_e \int_{y_1}^{y_e}\left(K_1 - \frac{y^2_e\, y_1}{2}\right) y\, dy$$

$$+ \ b_e \int_{y_1}^{y_e}\left(y^2_e - \frac{y^2}{3}\right) y^2\, dy = b_e\left(K_1 - \frac{y_e^2\, y_1}{2}\right)\left(y^2_e - y_1^2\right)$$

$$+ \frac{b_e\, y^2_e}{3}(y_e^3 - y_1^3) - \frac{b_e}{15}(y_e^5 - y_1^5) \ldots\ldots\ldots\ldots\ldots \quad (31)$$

For the area, A_2:

$$\int_0^y \frac{m\,dy}{b_1} = \int_0^y \left(\frac{m_1}{b_2} + \frac{y_1^2 - y^2}{2}\right) dy = y\left(\frac{m_1}{b_2} + \frac{y_1^2}{2} - \frac{y^2}{6}\right).$$

Now, $dA = 2\, b_2 dy$, hence the value of K for the area, A_2, will be:

$$\int_0^{y_1}\left(\int_0^y \frac{m\ dy}{b_1}\right) y\, dA = 2\, b_2 \int_0^{y_1}\left(\frac{m_1}{b_2} + \frac{y_1^2}{2} - \frac{y^2}{6}\right) y^2\, dy$$

$$= \frac{2\, m_1}{3} y_1^3 + \frac{b_2\, y_1^5}{3} - \frac{b_2\, y_1}{15} = y_1^3\left(\frac{2\, m_1}{3} + \frac{4\, b_2\, y_1^2}{15}\right) \ldots\ldots \quad (32)$$

Then the value of K is obtained by adding Equations 31 and 32.

$$K = b_e K_1 \left(y_e^2 - y_1^2\right) - b_e y_e^4 \left(\frac{y_1}{2} - \frac{4\, y_e}{15}\right)$$

$$+ \frac{y_1^3}{3}\left[2\, m_1 + \frac{b_e\, y_e^2}{2} + \frac{y_1^2}{5}(b_e + 4\, b_2)\right] \ldots\ldots\ldots \quad (33)$$

Application to Standard 12-in., 31.5-lb. I-Beam.—

$b_e = 5$ in.	$y_e = 6$ in.	$I = 215.4$	$m_1 = 15.47$	$K_e = 296.4$
$b_2 = 0.35$ in.	$y_1 = 5.46$ in.	$O = 35.9$	$K_1 = 322.7$	$K = 11\ 298$

The value of the error in the extreme fiber stress is then given at once by Equation 10: Mr. Williams.

$$e = \frac{dS\ E}{dx\ (215.4)\ G}\left[\frac{11\ 298}{35.9} - 296.4\right] = 0.0850\ \frac{dS\ E}{dx\ G}.$$

For the case of a steel beam fully loaded with a uniformly distributed load, $\frac{dS}{dx} = w$, and $\frac{E}{G} = 2.67$, as before, and

$$e = 0.0850\ w\ (2.67) = 0.227\ w.$$

Then, from Equation 11, the percentage of error will be:

$$e = \frac{22.7\ w\ (35.9)}{M} = \frac{815\ w}{M}.$$

Using the maximum bending moment, $M = \frac{w\ l^2}{8}$, at the center, the

$$\text{percentage, } e = \frac{6\ 520}{l^2}.$$

When $\frac{d}{l} = \frac{1}{10}$, $l = 10$ ft. $= 120$ in., and the percentage, $e = 0.45$.

When $\frac{d}{l} = \frac{1}{2}$, $l = 2$ ft. $= 24$ in., and the percentage, $e = 11.3$.

Application to Bethlehem 12-in., 28.5-lb. I-Beam.—A similar application of the foregoing formulas to this beam results in a value for the error in fiber stress of

$$e = 0.0937\ \frac{dS\ E}{dx\ G}.$$

This value is seen to be not quite 10% more than the error found for the standard beam.

Application to Girder Sections.—By a theoretical analysis similar to that used for the foregoing rolled sections, the following formulas have been derived, with the notation as shown in Fig. 16. The algebraic derivation need not be given, as it requires considerable space and can easily be checked by analogy with the preceding derivation. There is thus found:

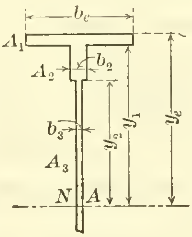

Fig. 16.

$$m_1 = \frac{b_e}{2}(y_e^2 - y_1^2), \text{ and } m_2 = \frac{b_2}{2}(y_1^2 - y_2^2)$$

$$K_e = K_3 - y_e^2\left(\frac{y_1}{2} - \frac{y_e}{3}\right) \quad \dots\dots\dots\dots\dots\dots (34)$$

$$\text{If } K_3 = \left(\frac{m_1}{b_2} + \frac{y_1^2}{2}\right)(y_1 - y_2) + k \quad \dots\dots\dots\dots (35)$$

$$\text{and } k = y_2\left(\frac{m_1 + m_2}{b_3} + \frac{y_2^2}{2}\right) \quad \dots\dots\dots\dots\dots\dots (36)$$

Mr. Williams.

$$K = b_e K_3 (y_e^2 - y_1^2) - b_e y_e^4 \left(\frac{y_1}{2} - \frac{4 y_e}{15}\right) + \frac{y_1^3}{3}\left[2 m_1 + \frac{b_e y_e^2}{2} + \frac{y_1^2}{5}\right.$$
$$\left.(b_e + 4 b_2)\right] + b_2 K_2 (y_1^2 - y_2^2) - b_2 y_1^4 \frac{y_2}{2} + \frac{y_2^3}{3}\left[2 m_2 + \frac{b_2 y_1^2}{2}\right.$$
$$\left. + \frac{y_2^2}{5}(b_2 + 4 b_3)\right] \dots\dots (37)$$

$$\text{in which } K_2 = k - \frac{m_1 y_2}{b_2} \dots\dots (38)$$

A check on these equations is obtained by letting $y_2 = 0$, for which case they should reduce to the corresponding equations for rolled sections. For instance, $K_2 = 0$, $K_3 = K$, and Equation 37 for K reduces at once to Equation 33 for K. When both y_1 and y_2 are zero, the formulas for the simple rectangular section should result. For this case, both Equations 37 and 33 reduce to $\frac{4\ b_e\ y_e^5}{15}$; or, as $b_e = b$ and $y_e = \frac{d}{2}$, to $\frac{bd^5}{120}$, agreeing with Equation 13. The most elegant mathematical form of derivation, of course, would be to derive the general equation first, and then find the special forms for the simpler sections by substitution, as suggested above. It seemed best, however, not to attempt to give the complete algebraic analysis for the general case, as it would probably appear rather formidable to the average practical engineer.

Application to the Author's Girder Section, 4A.—For this case there is found:

$y_e = 21$ in.	$b_e = 10$ in.	$m_1 = 205$	$I = 12\ 345$
$y_1 = 20$ in.	$b_2 = 1.25$ in.	$m_2 = 90$	$O = 587.8$
$y_2 = 16$ in.	$b_3 = 0.25$ in.	$k = 20\ 928$	$K_3 = 22\ 384$
$K_e = 21\ 061$	$K_2 = 18\ 304$	$K = 13\ 233\ 000$	

Then Equation 10 gives: $e = 0.1176 \frac{dS}{dx}\frac{E}{G}$.

For the case assumed by the author, $\frac{dS}{dx} = w$, and $\frac{E}{G} = 2.67$; for steel, therefore: $e = 0.314\ w$.

Then Equation 11 gives:

$$\text{the percentage, } e = \frac{31.4\ w\ (587.8)}{M} = \frac{18\ 457\ w}{M}.$$

For the maximum bending moment at the center, $M = \frac{w\ l^2}{8}$, and there results:

$$\text{the percentage, } e = \frac{147\ 660}{l^2}.$$

Mr. Williams.

When $l = 332$ in., the percentage, $e = 1.34$ *versus* 2% in Table 1.

When $l = 420$ in. $\left(\frac{d}{l} = \frac{1}{10}\right)$ the percentage, $e = 0.84$ *versus* 1.24% in Table 1.

Application to Author's Girder Section, 4D.—For this case there is found:

$y_e = 20.5$ in.	$b_e = 10$ in.	$m_1 = 101$	$I = 12\,037$
$y_1 = 20$ in.	$b_2 = 2$ in.	$m_2 = 144$	$O = 587.2$
$y_2 = 16$ in.	$b_3 = 1$ in.	$k = 5\,968$	$K_3 = 6\,970$
$K_e = 5\,638$	$K_2 = 5\,160$	$K = 3\,677\,000$	

Then Equation 10 gives $e = 0.0518 \dfrac{dS}{dx}\dfrac{E}{G}$.

As before, let $\dfrac{dS}{dx} = w$, and $\dfrac{E}{G} = 2.67$, therefore $e = 0.138\,w$.

Then Equation 11 gives:

$$\text{the percentage, } e = \frac{13.8\,w\,(587.2)}{M} = \frac{8\,120\,w}{M}.$$

When $M = \dfrac{wl^2}{8}$, there results:

$$\text{the percentage, } e = \frac{64\,960}{l^2}.$$

And when $l = 88$ in., the percentage, $e = 8.39$ *versus* 11% in Table 1.

" $l = 410$ in. $\left(\frac{d}{l} = \frac{1}{10}\right)$, the percentage, $e = 0.39$ *versus* 0.56% in Table 1.

Maximum Values of Error, e.—As the error in extreme fiber stress, *e*, is a direct function of the change in shear, the maximum error will be found for the case where heavy concentrated loads are carried by the beam, as already discussed. In order to ascertain the approximate value of the error for such cases, consider the simple case of a beam supporting a concentrated load at the center, and assume such load to be uniformly distributed over a short assumed distance, $2a$. Then the maximum shear will be $S = wa$, and the maximum bending moment $M = wa\dfrac{l - a}{2}$, if $w =$ the assumed uniformly distributed load per inch $= \dfrac{\text{total load}}{2\,a}$.

Let $f =$ the allowable intensity of bending fiber stress; and $s =$ the allowable intensity of shearing stress.

Assuming a beam designed for equal strength in bending and in shear, the value of w at once results from the allowable shear, and then the theoretical length, l, from the bending moment equation.

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Rectangular Sections.—For rectangles, the maximum intensity of shearing stress occurs at the neutral axis, and is 1.5 times the average shear. Therefore

$$s = 1.5 \frac{S}{bd} = 1.5 \frac{wa}{bd}, \text{ whence:}$$

$$w = \frac{s\,bd}{1.5\,a} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (39)$$

$$\text{Then } M = wa \frac{l - a}{2} = s\,b\,d \frac{l - a}{3} = f\,O = f \frac{bd^2}{6}, \text{ whence:}$$

$$l = \frac{f\,d}{2\,s} + a \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (40)$$

For steel rectangles, by Equation 14, as $\frac{dS}{dx} = w$, and $\frac{E}{G} = 2.67$, the value of error, e, would be, using Equation 39:

$$e = 0.267 \frac{w}{b} = 0.178 \frac{s\,d}{a}.$$

Then the percentage of error would be

$$e = \frac{100\,e}{f} = 17.8 \frac{s\,d}{f\,a} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (41)$$

The author assumes the ratio of $\frac{s}{f}$ to be $\frac{3}{4}$, therefore Equation 41 would give:

$$\text{the percentage, } e = 13.33 \frac{d}{a} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (41a)$$

The percentage of error would then vary directly with the depth of the rectangular section and inversely with the half distance over which the concentrated load is assumed to be distributed, and becomes very large for deep short beams. For instance, if $d = 24$ in. and $a = 4$ in., Equation 41a gives the percentage, $e = 80$.

The length, l, would be 20 in., from Equation 40, and it is seen that the error is independent of the breadth, b, of the rectangle, which could be made sufficient to develop any required bearing area at the load or at the supports.

Rolled and Girder Sections.—Assuming for these sections that the allowable shear intensity, s, is uniformly distributed over the gross area of the web, A_w, the value of w would be:

$$s = \frac{S}{A_w} = \frac{wa}{A_w}, \text{ whence } w = \frac{s}{a} A_w \quad \ldots\ldots\ldots\ldots \quad (42)$$

$$\text{Then } M = wa \frac{l - a}{2} = s\,A_w \frac{(l - a)}{2} = f\,O, \text{ whence}$$

$$l = \frac{2\,f\,O}{s\,A_w} + a \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (43)$$

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For the standard 12-in., 31.5-lb. I-beam, the value of e was found to be:

$$e = 0.227\ w.$$

Then the percentage, $e = \frac{100\ e}{f} = \frac{22.7\ w}{f} = 22.7 \frac{s\ A_w}{f\ a}$ . . (44) from (42)

For $\frac{s}{f} = \frac{3}{4}$, the percentage, $e = 17.0 \frac{A_w}{a}$ . . . . (44a)

For this beam, $A_w = 4.2$ sq. in., and, if $a = 4$ in., $e = 17.9$ per cent.

The length would be 50 in. by Equation 43.

A similar computation for the Bethlehem 12-in., 28.5-lb. I-beam gives:

$$\text{the percentage, } e = \frac{25.0\ w}{f} = 25.0 \frac{s\ A_w}{f\ a}.$$

For $A_w = 3$ sq. in., and $\frac{s}{f} = \frac{3}{4}$, assuming $a = 4$ in., $e = 14.0$ per cent.

The girder section, $4A$, gives:

$$\text{the percentage, } e = \frac{31.4\ w}{f} = 31.4 \frac{s\ A_w}{f\ a} \qquad (45)$$

For this case, $A_w = 10.5$ sq. in. Let $a = 6$ in. and $\frac{s}{f} = \frac{3}{4}$, then $e = 41$ per cent.

Similarly, girder section $4D$ gives:

$$\text{the percentage, } e = \frac{13.8\ w}{f} = \frac{13.8\ s\ A_w}{f\ a} \qquad (46)$$

$A_w = 41$ sq. in., $\frac{s}{f} = \frac{3}{4}$, and assume $a = 6$ in., as above, then $e = 70.8$ per cent.

Of course, w is very large (82 000 lb. per in.) and $l = 44$ in., from Equation 43, requiring heavy stiffeners at the load and the supports. This girder, being exceptionally strong in shear, on account of the heavy 1-in. web, the error introduced by neglecting shear distortion for such a special short beam is seen to be exceedingly large. Practically, it is much less, as the heavy stiffeners under the load would tend to prevent the large shear distortion on which the error is based.

The above values are not exact on account of the assumed value for s as the average value on the web A_w, whereas it should be the maximum value at the neutral surface from the equation, $s = \frac{S\ m}{b_1\ I}$. Instead of A_w in these equations, $\frac{b_1\ I}{m}$ might be used, giving results from 10 to 20% less. Since it is the usual practice to design girder

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sections by the use of average, s, on web, A_w, the formulas and results are given in that form.

For Ordinary Design, with $\frac{d}{l} = \frac{1}{10}$.—Assuming the design load to be fixed by the bending fiber stress, f, the value for w would be

$$M = wa \frac{l - a}{2} = f\,O, \text{ whence } w = \frac{2 f\,O}{a\,(l - a)}.$$

For rectangular sections,

$$\text{the percentage, } e = 8.88 \frac{d^2}{a\,(l - a)} \quad \ldots\ldots\ldots\ldots\ldots \quad (47)$$

For the standard 12-in. I-beam, $e = 0.227\ w$, and

$$\text{the percentage, } e = \frac{22.7\ w}{f} = \frac{45.4\ O}{a\,(l - a)} \quad \ldots\ldots\ldots\ldots \quad (48)$$

If $l = 10$ ft. $= 120$ in., and $a = 4$ in., then the percentage,

$$e = \frac{45.4 \times 35.9}{4\,(120 - 4)} = 3.5 \text{ per cent.}$$

The Bethlehem 12-in. I-beam gives:

$$\text{the percentage, } e = \frac{25.0\ w}{f} = \frac{50.0\ O}{a\,(l - a)} \quad \ldots\ldots\ldots\ldots \quad (49)$$

For $l = 120$ in., $a = 4$ in.; $e = 3.88$ per cent.

The girder section, $4A$, gives:

$$\text{the percentage, } e = \frac{31.4\ w}{f} = \frac{62.8\ O}{a\,(l - a)} \quad \ldots\ldots\ldots\ldots \quad (50)$$

For $l = 420$ in., $a = 6$ in.; $e = 14.9$ per cent.

The girder section, $4D$, gives:

$$\text{the percentage, } e = \frac{13.8\ w}{f} = \frac{27.6\ O}{a\,(l - a)} \quad \ldots\ldots\ldots\ldots \quad (51)$$

For $l = 410$ in., $a = 6$ in.; $e = 6.7$ per cent.

Conclusion.—The foregoing theoretical analysis indicates, more completely than Mr. Prichard's analysis, that the results obtained by the common theory of flexure for maximum bending fiber stress may be in error by large percentages on the side of danger. For certain special cases of large concentrated loads on short spans the error may possibly be as high as from 50 to 70% of the value found by the common theory. These high values are not actually correct, however, because the effect of stiffeners, and the secondary effect of such errors on the theoretical shear values have been neglected; yet it is probably true that all designs of short heavy beams carrying concentrated loads are introducing large errors when based on bending fiber stress. Ordinary bridge pins would be in this class, and the high allowable fiber stresses and high results of tests of short round beams would seem like

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an anomaly, in the light of the foregoing theory. The fact that such short beams are really restrained at the ends and act almost wholly with direct compressive arch resistance will probably account for the anomaly.

As the foregoing analysis was based on error in extreme fiber stress, the percentages obtained are not strictly comparable with those obtained by the author in Table 1, based on error in resisting moment or capacity. It seems more logical to use the actual design quantity, that is, the extreme fiber stress, as the basis for error; and the results for such analysis are seen to be consistently less than those found by Mr. Prichard. The errors are seen to be less for standard rolled I-beams than for Bethlehem beams, and greater for girders with thin webs for ordinary design lengths—say when $\frac{d}{l} = \frac{1}{10}$. The vital effects of the ratio of $\frac{E}{G}$ and of change in shear at concentrated loads are clearly shown, as well as the effect of the ratio of length of span to depth of beam.

The speaker desires to emphasize the value of Mr. Prichard's paper in calling attention to the practical meaning of the work of the mathematical theorists, such as Love, Lamé, Saint Venant, and others. A thorough knowledge of the practical possibilities and limitations of given theoretical formulas is essential to any intelligent use of theory, and if the paper and discussion result only in calling attention to these practical limitations, and errors in the common theoretical formulas and methods of beam design, they will be of great value.

It is hoped that the theoretical discussion here given may be helpful to the author in the preparation of his proposed paper on plate girder analysis and design. There are, however, limitations or modifications of the theory which require further study: first, the actual effect of stiffeners in distributing concentrated loads to the web, or the actual web stresses under such concentrated loads when no stiffeners are provided; and second, the actual distribution of flange stress in flanges composed of cover-plates riveted to flange-angles. The foregoing theories certainly do not apply to such sections, as the slip at working loads, which occurs in riveted work, means a horizontal shear weakness, and would certainly indicate the probability that the outside cover-plates never get their computed value of intensity of stress. That such slipping occurs in riveted work at working intensities has been shown by recent tests.* If such slip of the riveted connections between cover-plates and flange-angles and between flange-angles and web occurs, all theories would be directly affected.

* Reported in *Bulletins Nos. 62* and *125*, American Railway Engineering Association; also *Bulletin No. 49*, University of Illinois.

Mr. Marburg.

EDGAR MARBURG, M. AM. SOC. C. E. (by letter).—This paper contains the first public announcement of the results of comparative tests made at Ambridge, Pa., some two years ago, on 84 Standard and Bethlehem I-beams, representing in the aggregate nearly 30 tons of materials. In the writer's opinion, the interest of the paper, from a practical view-point, centers in this particular feature, to which he intends to restrict his discussion.

The writer regards it as highly regrettable that tests of such great interest to the Profession, and affecting, moreover, conflicting commercial interests, should have been reported so meagerly. Referring first to features, the availability of which is beyond question: the structural details—namely the connection angles at the ends and through the web—should have been described and preferably illustrated; the manner in which the loads were imparted to the beams, and the means used for determining the loads at predetermined permanent sets should have been fully explained; and lastly, the results of each individual test should have been cited, for the obvious reason that the range of values may be highly significant. In fact, generally speaking, a knowledge of the lowest value of a member subjected to service as a separate unit, is of more practical importance than a knowledge of the average value of a series of such members.

The report of these tests, however, is fairly open to much weightier criticism, in its omission of numerous features of fundamental importance, scientifically as well as practically. The writer does not know whether these omissions are chargeable to the unnamed authors of the tests, or to the author of the paper. To some of these features the writer intends briefly to allude.

1.—Why should the tensile tests have been confined to material from the flange and web, to the apparent exclusion of the root? The tests by the writer, to which the author has referred, and other tests, have shown that the material at the root of the flange (as represented by specimens of the thickness of the web and of a width somewhat in excess of the thickness of the flange at the center) is apt to be of a wholly abnormal character. Unfortunately, this inferior material is thus most likely to exist at that part of the flange where its effect on the strength of the beam is greatest.

2.—Why was the tensile elastic limit, by extensometer measurements, not determined, in addition to the yield point, by the drop of scale beam? It has been abundantly proven that, in so far as the inferiority of steel may be revealed by an abnormally low elastic limit, the yield point gives little or no indication of that condition. Thus, in the writer's tests, in which the elastic limits were found by means of a Ewing extensometer, the elastic limits of the material from the root, in four cases out of seven, ran as low as about one-third to six-tenths of the yield point, the latter not showing much

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variation from what may be regarded as its normal value. In these four instances the elastic limit of the material in the web exceeded that of the material in the root of the same beam by 195, 126, 99, and 61%, respectively. In three of these cases the specimens had been cut from Bethlehem beams, and in the fourth—for which the percentage of difference above referred to was 61—from a Standard beam. In the absence of extended comparative tests, it is idle, therefore, to discuss the relative uniformity in quality of Bethlehem and Standard beams. The writer's views on this phase of the subject were expressed in his paper on his own tests, as follows:

"While there seems to be some justification for the claim that Bethlehem beams possess greater freedom from initial stresses than standard beams, experimental proof is needed for its actual demonstration.

* * * * * * *

"The claim of superior uniformity in the physical properties of the material throughout the section, advanced in favor of the Bethlehem beams, is not substantiated by these tests. Of the six Bethlehem beams, in which the material was subjected to tensile tests, the uniformity with respect to the elastic limit was good in one case, fair in another, and bad in the remaining four. * * * The writer regrets that he was not afforded the desired opportunity of making numerous additional tests of this character. In the light of these results it would seem that the material in the vicinity of the root received insufficient work, or that it was finished at too high a temperature, or both, in the Bethlehem as well as the Standard beams."

3.—The reported bending tests contain no data as to elastic-limit values. The reference to permanent sets shows that deflections were observed, although no intimation is given as to the apparatus and methods used therefor. Why, then, are the elastic-limit loads not reported? The "elastic-limit load" may perhaps most reasonably be defined as that load above which the load-deflection ratio ceases to remain constant. In the writer's tests, this load was usually found to be greater, and in some cases much greater, than the load at which permanent set (0.01 in.) was first observed. For the values of the latter one also looks in vain in the present paper.

In the writer's tests, the computed extreme fiber stresses at the elastic-limit loads were found to be very nearly constant for the Standard beams, the average value lying slightly above 20 000 lb. per sq. in. for 15- and 24-in. beams loaded centrally and at the quarter points.

For the Bethlehem beams these computed stresses fell off very rapidly, however, with increasing beam depth. The average values for 15-, 24-, and 30-in. depths were 25 700, 18 200, and 12 100 lb. per sq. in., respectively. The remarkably low value for the 30-in. beams represents the average of four tests, of which the lowest value was

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10 800 lb. per sq. in. As conclusive proof that this extraordinarily low value is not to be attributed to twisting action, defective mountings, or other extraneous causes, it may be said that the beam in question was loaded at the quarter points and that the extreme fiber deformations were carefully measured with four Johnson dial extensometers reading to 0.001 in. on a gauge length of 40 in. between the points of application of the load. The readings of each pair of extensometers on opposite sides of the same flange were substantially in agreement for the entire range of observations, thus showing conclusively the absence of lateral deflections of the flanges. The average elastic limit of the flanges, based on these direct fiber deformations, was 11 100 lb. per sq. in., that is, within 300 lb. per sq. in. of the computed elastic limit in bending for the beam as a whole. In final corroboration, it may also be said that this beam developed its first permanent set of 0.01 in. at the load corresponding to the computed extreme fiber stress of 10 800 lb. per sq. in., above which the load-deflection ratio ceased to be constant.

Relative to this matter, it may also well be asked: why were extreme fiber deformations not determined (or at least not reported) in connection with the tests adduced by the author?

4.—It is difficult to understand why the author should have considered it worth while to report the loads at permanent sets of 0.4 in.; and it is still more difficult to understand why he should not have thought it worth while to report the maximum loads before rupture. It is to be confidently anticipated that the latter could not have been much higher than the former, but why are their values not given?

Mr. Godfrey, in his reference to the writer's tests, states that he has "called attention, a number of times, to the fact that the results of these tests are practically valueless, because the beams had no lateral support whatever." The writer is aware that Mr. Godfrey has been an ardent propagandist against his tests, from the time that they were first reported. On one occasion only has he seen fit to reply, categorically and *in extenso,* to Mr. Godfrey's criticisms. Those who are sufficiently interested in the subject will find this reply in one of the engineering journals.* The writer will content himself here with calling attention to the absolutely false description by Mr. Godfrey of the behavior of the beams in question under test, and the assumed effect of that imaginary action on the reported elastic-limit values. The answer is practically contained in the following excerpt from the paper itself:

"In the earlier tests the attempt was made to observe the lateral deflection of the upper flange. * * * It was found that slight, irregular movements occurred during the early stages of loading while

* *Engineering-Contracting*, November 3d, 1909, p. 385.

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the beam was apparently adapting itself to its bearings. No subsequent movements were then discernible until the elastic limit of the beam had been exceeded, and usually not until the ultimate strength had been nearly reached. * * * Since these measurements were very time-consuming, and appeared to furnish little or no useful information, they were omitted in the later tests."

Notwithstanding this, Mr. Godfrey falsely attributes the reported low elastic limits to the twist in the web, owing to the absence of lateral supports at the ends. He speaks of the effect on the deflection of the versed sine of an arc described above the support by the middle point of the web. As a matter of fact, no angular movement whatever took place until the elastic limit of the beam had been far exceeded and deflection readings had therefore been discontinued. Mr. Godfrey's arc is therefore purely imaginary.

The writer is on record in the opinion that the elastic limit in bending would not be increased by changes in the end supports or by lateral bracing, in comparison with the conditions under which his tests were made, and he has seen no sound reason for changing that opinion. The indications of these tests cannot be refuted by mere argument. If the conclusions to which they point, based on painstaking observations the results of which have been fully reported, are incorrect, that fact can be established in only one way, namely, by tests of full-sized beams, and not by Mr. Godfrey's toy models of paper and tin.*

There can be no reasonable doubt that if the extreme fiber deformations had been observed in the tests reported by the author, it would have been seen that the elastic limit of the material in the flanges was passed before the loads corresponding to permanent sets of 0.1 in. were reached. Whether extreme fiber stresses in excess of the elastic limit are to be regarded as necessarily dangerous under conditions in which permanent set *per se* is not objectionable, the writer does not venture to affirm with confidence; nor does he know of any experimental evidence which can be looked to for a conclusive answer to that question. It is true that the primitive elastic limit of the material in the flanges is raised by such action, followed by a period of rest, but that is no less true of material subjected to pure tension. It might just as well be argued—as has in fact been done—that the quality of eye-bars may be improved by raising the elastic limit of the material under initial stresses in excess of the elastic limit. In the absence of conclusive experimental proof, the manner in which observed phenomena of this kind are to be interpreted must be left to individual judgment. It is essential, however, that such phenomena should be accurately observed and faithfully reported in any effort designed to contribute to the real advancement

* *Engineering News*, January 6th, 1910.

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of knowledge on this subject. In the light of present knowledge it may reasonably be assumed that few engineers would knowingly adopt a working stress for the extreme fibers of deep I-beams in excess of the elastic limit of the material, even for conditions of loading not strongly conducive to ultimate failure by fatigue.

Three years have now elapsed since the publication of the results of the writer's tests. If the results of these tests, especially with respect to their most important feature—namely, the elastic limit—are incorrect, it would have been an easy matter to have proved this long ago by means of full-sized tests. It may not be generally known to the Profession that such full-sized tests, with accompanying minute observations on beams in which twisting action was prevented by substantial cast-iron yokes to which the beams were clamped at the supports, have actually been made. These tests were conducted by the Bethlehem Steel Company for several months immediately after the results of the writer's tests were publicly announced. The writer is not aware, however, that the results of these manufacturers' tests have been published. It is to be hoped that the author's paper will serve to bring these records to light. The Engineering Profession is manifestly entitled to a knowledge of the facts, whether favorable or unfavorable to the product involved. In any broad view, the manufacturers should be equally willing to have such facts brought out fully and fairly by means of scientifically conducted tests under wholly disinterested auspices. Unfortunately, the tests meagerly reported by the author contribute comparatively little information of value.

In conclusion, the writer desires to quote the closing remarks from his earlier reply to Mr. Godfrey:

"More light on this subject can come only from further tests and not from grossly prejudiced discussions, based on distorted references to carefully observed and fully recorded facts. It is to be hoped that manufacturers, as well as engineers, will be brought to share this reasonable view, and that they will be moved to welcome and to promote further investigations of their product at the hands of those whose interest in the subject lies only in their desire to establish 'the truth, the whole truth and nothing but the truth'."

Mr. Prichard.

Henry S. Prichard, M. Am. Soc. C. E. (by letter).—Referring to Mr. Stern's discussion, it is gratifying to have the portions of the paper relating to the use of channels as beams, and to the initial internal stresses in I-beams, confirmed by Mr. Stern's experience.

In regard to his query as to the strength of a 20-in., 100-lb. I-beam, 6 ft. long: By reasoning similar to that used in computing Table 1, the uniformly distributed load which would produce a given intensity of stress within the elastic limit in the extreme fibers of the beam cited, supposing it to be initially free from internal stresses, would be about 4% less than indicated by the theory of flexure. After the elastic limit is passed, however, the beam may develop additional permanent resistance by taking a slight permanent set, as described in Section 6 of the paper. The load for such a short span would probably be concentrated instead of uniformly distributed. If the loads are concentrated, the points of concentration and the manner of applying the loads would have to be specified before Mr. Stern's query could be answered. Mr. Prichard.

The tests of the beams cited in Table 4 were part of a series of tests of beams, beam connections, and plate girders undertaken by the Carnegie Steel Company, the Illinois Steel Company, and the American Bridge Company, primarily for their own information, and conducted by a committee composed of James H. Edwards, John C. Neale, and John Brunner, Members, Am. Soc. C. E.

For the purpose of making the tests, the 2 000-ton eye-bar testing machine at the Ambridge plant of the American Bridge Company was reconstructed by fitting a framework against the main longitudinal girders, as shown by Fig. 17. The pulling arrangement consisted of an equalizing device pulled by an eye-bar connected to the cylinders of the eye-bar testing machine, and carrying at its ends shackles, by the adjustment of which loads could be applied at different points. The construction of this testing machine required the placing of beams in a horizontal position, but, as the beams were supported and guided at intervals of one-third of the length, the bending moment in the vertical direction became a minimum, and was disregarded. The apparatus was arranged so that either one central load or two equal loads, symmetrically placed at varying distances apart, could be applied to the beam. The loads were applied by small increments, in accordance with schedules suited to the depth of the beam, its length, and the condition of loading. The sets and deflections under loading were read on circular deflectometers, which magnified the movement of the beam so as to permit of close readings. One of these deflectometers was placed at the center of the beam, or as closely thereto as possible, and two others at approximately equal distances on each side of the center, and usually as nearly as possible at the third points of the beam spans.

From the high standing of the engineers who conducted the tests, as well as from the purpose for which the tests were made, the writer is confident that every reasonable precaution was adopted to make them truly informative, actually and relatively.

The depths of the beams given in Table 4 were selected for the reason that they are those most commonly used in ordinary building construction. The lengths used in these tests were selected as a fair average of the spans for which these beams are used. The conditions of loading and supports were those usual in actual construction, and were as described on page 917.

Mr. Prichard.

Referring to Mr. Worthington's discussion, if the theory of flexure was strictly accurate, it would apply rigidly to both steel and soft rubber. In a strict sense, however, as shown in the paper, it is only an approximation, though a remarkably close one within wide limits. As a theory, it has its faults, which, naturally, are more manifest in soft rubber than in steel. For this reason, soft rubber was chosen to exhibit the fact that originally straight cross-sections deform in a reverse curve, somewhat like a long $\int$, instead of remaining straight, as assumed in the theory. In the case of the rubber beam exhibited, this deformation is partly due to the local effects of concentrations, which local effects, as pointed out in Section 2 of the paper, are not taken into account in the ordinary theory of flexure; hence the rubber beam really illustrates a combination of faults, described in Sections 1 and 2, instead of illustrating only the fault described in Section 1; and, though suggestive, is not in itself conclusive evidence of the fault described in Section 1. A demonstration of this fault, however, is given by the noted authority, Professor C. Bach,* and this fact is cited in the paper, in which also an independent proof is given. Mr. Worthington, who denies that there is any similarity between "this rubber beam" and "an actual beam of steel," does not mention either of these demonstrations, and it does not appear, from his discussion, whether he rejects (without argument) the fact demonstrated, or accepts the fact, namely, that originally plane cross-sections do not remain plane during flexure, and only objects to the means used to illustrate it.

In discussing this fact, Mr. Williams states:

> "That, under certain conditions, there is an error in the usual common theory of plane normal sections, is at once evident when the facts of shear distortion or strain are considered, as is indicated by the author in Fig. 3."

He further states: "This fundamental assumption of the common theory is never precisely true, as all normal sections are curved by shear distortion"; and Mr. Godfrey states that the "paper demonstrates clearly the inapplicability of the common theory of flexure to very short, deep beams and girders."

In Section 2 it is shown that the ordinary theory of flexure neglects the local effects of concentrated loads and reactions. One of these effects is a tendency to buckle the web.

In the standard I-beam sections, adopted by the Association of American Steel Manufacturers in 1896, the webs are of such thickness that they are rarely the weakest point; in consequence, architects and engineers have given the webs little consideration, and have very seldom used stiffeners.

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* "Elastizität und Festigkeit," p. 459.

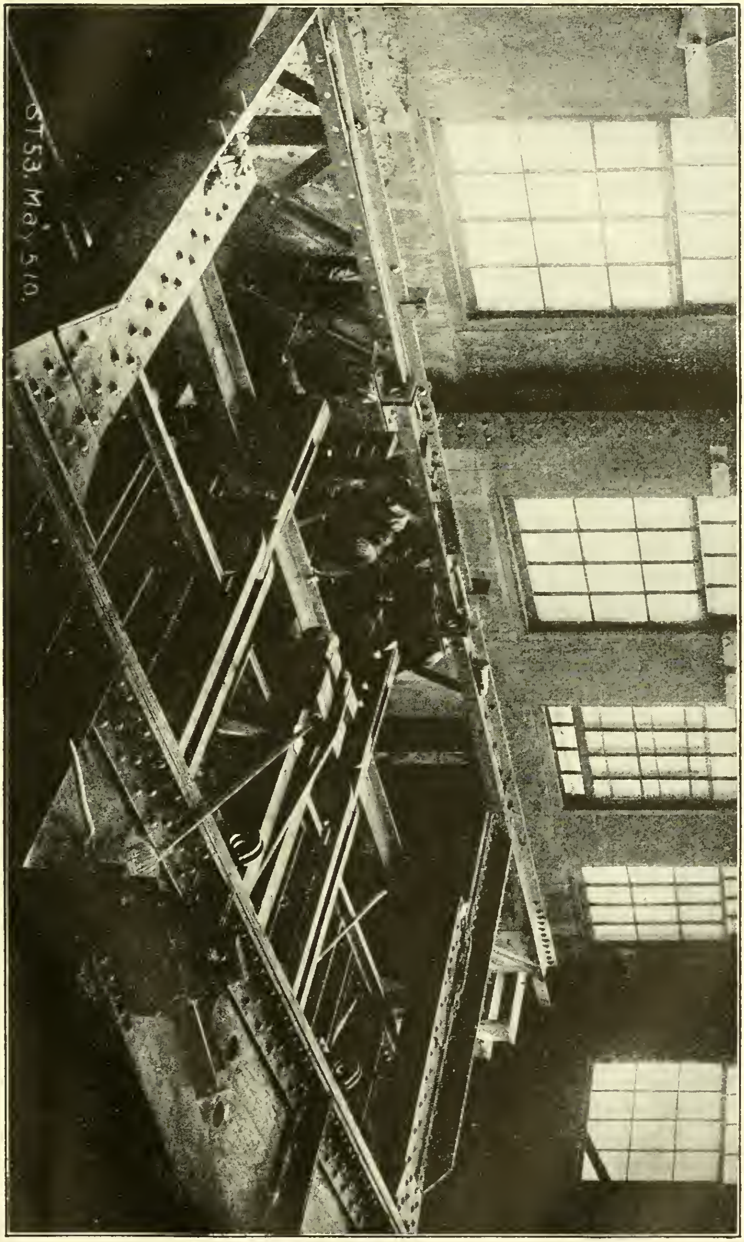

FIG. 17.—BEAM TESTING MACHINE, AS ARRANGED FOR THIRD POINT LOADING.

Mr. Prichard.

As beams with thinner webs than the standard are now rolled, the paper calls the special attention of architects and engineers to the fact, and points out that thin webs have less resistance to buckling than thick ones, and that, unless the web is reinforced, it has to be strong enough to resist both the shear and the concentrations, where there are any.

Even when there is no danger of buckling, it is an open question whether it is true economy to use beams with thinner webs than those of the standard beams.* Whether, when the web is too weak safely to stand the concentrations, it is better to use beams with thicker webs or to add "stiffeners milled to bear against the flanges and riveted directly to the webs of the beam," as recommended by Mr. Worthington, depends on the conditions.

Mr. Worthington can see no reason, in theory or in fact, for making any distinction, as regards concentrations, between standard beams and those with thinner webs, but he adds the general caution that "the designer should always consider the matter of how each concentrated load shall be applied to the beam he is designing, whatever its section."

It is to be hoped that those who are inclined to heed the caution as to thin webs will not be misled by Mr. Worthington's discussion into the belief that there is only one company rolling beams with thinner webs than standard. There are, in the United States, at least two companies which roll such beams, and the practice may become general. The point with which the paper deals in this regard is the shapes of the beams. The names of the concerns rolling them are not material to the engineering questions discussed.

The caution as to thin webs occurs in Section 2. To Mr. Worthington, this section, three-fourths of which are entirely general, seems to be merely an invidious comparison between standard I-beams and those rolled by a particular company, and he makes the same mistake regarding the "whole gist" of the paper. To him the portions which deal with any theme other than the comparative value of the said beams, or which have any broader application, are few and unimportant.

The writer's intention to publish his investigations of faults in the theory of flexure was formed, and much of the matter published existed in a fragmentary state among his papers, before there was, in the United States, at least, any departure from the standard I-beams, in the way of thinner webs and flanges.

The manuscript, with the exception of the references on the last three pages to manufacturers' tests of steel I-beams, was written, almost exactly as presented, before the writer had any information as to the results of those tests, and it would have been published had he never heard of them. The general report of those who conducted the tests

* In writing the above, transverse strength was the consideration in mind, but the condition of an I-beam just disclosed by the excavation for the removal of "The Hump" in Pittsburgh suggests other considerations. The beam is 8 in. deep and 8½ ft. span, and supported a sidewalk. Its web has entirely rusted through from connection angle to connection angle, leaving no connection whatever between the flanges for 8 ft.

Mr. Prichard. refers to, but does not incorporate, records of deflections and permanent sets of each beam as successive loads were applied during the progress of its test (the writer has not examined these records), and it gives diagrams showing the averages of the permanent sets observed from 0 to 0.4 in. for each size of beam, for each span, for each condition of loading.

From these graphic diagrams of permanent sets, as the paper states, the writer prepared Table 4, in which he endeavored to epitomize the results shown by the diagrams, by scaling the loads which produced permanent sets of 0.1 in. and 0.4 in. and converting the amounts scaled into multiples of the working load, *W*.

Those familiar with the preparation of such diagrams realize, of course, that it is not necessary to know in advance just what loads will produce given permanent sets, as assumed by Mr. Worthington. Instead, the method is to note the sets under successive loads, and connect the points indicating such sets by lines. Believing that Table 4 would be of some interest, that the large loads required to produce small permanent sets were reassuring, after the poor showing made in Professor Marburg's tests, and that it was pertinent to a paper on flexure which criticized the adoption of new shapes of I-beams without first testing them, permission to publish this table in the paper was sought and obtained, notwithstanding that it was pointed out at the time that the paper advocated repeated and endurance tests as a scientific means of determining permanent capacity, and would make comment regarding the table accordingly.

The tests and report thereon were not made with any regard to the paper, and so, of course, no special observations were made for the purpose of illustrating it and testing its author's contentions, as Mr. Worthington by his questions implies should have been done; in fact, those who made the tests and the report had no knowledge that such a paper was in preparation or contemplated. On the other hand, the paper was not written for the purpose of publishing the tests.

Mr. Worthington makes two contentions relative to including in the paper the manufacturers' tests of I-beams. He leads up to the first contention by a supplement of his own to the theory of flexure which involves the undemonstrated and fallacious proposition that the loads which produce slight permanent sets, say 0.1 in. (in I-beams of the same depth and moment of inertia), are inversely proportional to the flange widths. He concludes, with regard to the beams in Table 4, that theoretically it should take 21 and 22% more load to produce a slight permanent set in the standard than in the new beams, and, after comparing this result with the 18.6% difference shown by experiments in producing 0.1 in. of permanent set, he contends:

"Certainly such close agreement between theory and tests should bar out this table from a paper intended to demonstrate faults in that theory."

Mr. Prichard.

Mr. Worthington, before giving his second contention, points out the additional data which he considers should have been included in the table, and alleges that either those who made the tests were incapable or the data were purposely withheld, and he then contends:

"For surely it is not a tenable hypothesis that the author did not appreciate the overwhelming importance to his paper of including all these measurements in his record of those tests."

Why it is of overwhelming importance to a paper to include certain data in a table which should certainly be "barred out" of that paper Mr. Worthington does not explain.

The writer protests against Mr. Worthington's claim that the theory of flexure indicates that it takes 22 and 21% more load to produce a slight permanent set in 12- and 15-in. standard I-beams than it does to produce the same set in the corresponding beams of new shapes. It is surprising that one who objects so strongly to having greater resistance to concentrated loads claimed for the webs of the standard beams, on account of their greater thickness, should claim much greater theoretical resistance to permanent set for standard beams than for those of the new shapes shown in Fig. 10C. Of course, there must be some good reason for the smaller resistance to permanent set shown in Table 4 for the new shapes, but it has not been accounted for by the theory of flexure, and can be only partly explained, if at all, by Professor James Thomson's theory of overstrained beams (endorsed by his brother, Lord Kelvin). Possibly, it may be due, in part at least, to the tendency, which naturally develops as beams deflect, of the edges of the flanges to lag behind their centers in taking their share of the strain; a tendency which is naturally greatest in wide thin flanges.*

The principle of the first contention, namely, that facts which tend to confirm a theory should certainly be omitted from a paper intended to demonstrate faults in that theory and show its limitations, is wholly wrong. Engineers, in their papers and discussions, should not be partisan advocates, or attorneys pleading some client's case, but sincere contributors to engineering knowledge who endeavor to be absolutely fair.

The second contention overstates the importance, to a paper on faults in the theory of flexure, of complete details of the tests recorded in Table 4. It is doubtless a fact, however, that a paper which dealt exclusively with I-beams and gave complete details of these tests would be of greater immediate interest to engineers than the paper under discussion. The writer would be pleased to have such a paper presented to the Society by the engineers who conducted the tests, and whose efforts he rates very highly.

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* The edges of the top flange tend to buckle downward and the edges of the bottom flange to straighten upward, as a beam deflects, and thereby relieve themselves of stress, and they should thus relieve themselves to the extent permitted by the cross-sectional stiffness of the flanges, thus compelling the centers of the flanges, which cannot relieve themselves in that way, to take more than their share of the stress.

Mr. Prichard. The paragraph in the paper which called forth Mr. Worthington's statement as to the writer's opinion of those who conducted the tests is as follows:

"On an average, it took 18.6% more load to produce a permanent set of 0.1 in. in the beams of standard shape than in the nominally equivalent beams of new shapes, and 8% more to produce a permanent set of 0.4 in. Whether or not this indicates a corresponding superiority in permanent capacity, what the permanent capacities are, and what permanent sets the beams would take under their maximum permanent loads, are questions to be decided by scientific experiments."

Those who conducted the tests followed the very general practice of making but a single test of each piece. The writer favors repeated and endurance tests for determining maximum permanent capacity. If he had a poor opinion of those who follow the method of making a single test of each piece, it would have to include nearly the entire Profession.

The paragraph above quoted contains the only comment in the entire paper on the results of the tests given in Table 4, yet Mr. Worthington uses the phrase: "the author's 18.6% of implied advantage," and states that the author assumes such a superiority to be shown by the data published in his paper. Mr. Worthington must have derived his impression as to superiority of the standard beams over those of new shapes from the results of the tests themselves and not from any comment by the writer.

Mr. Worthington overestimates the comparative importance of ultimate strength, deflections, and measured strains; he greatly underestimates the importance of permanent sets, which he regards as "comparatively valueless," and he expresses a purely theoretical and wholly inadequate conception of elasticity and its limits, as applied to materials of construction.

"Elasticity is that property of matter by virtue of which a body will not change in bulk or shape except by force, and will recover its original bulk or shape on the removal of the force."*

For the purposes of the structural engineer, it is the most useful property of matter; the one on which he relies for the permanent strength of the structures he designs, and on which he bases his theories of deformation, and distribution of stress.

It would seem that all that is necessary is to commit to memory the theories, to ascertain the limits to elasticity of the materials of construction, and to take a course in mathematics; then structural engineering becomes a mere matter of computation. This is the conception which students in structural engineering are apt to derive from their textbooks, and which some carry with them into their engineering practice. It is a beautiful conception, and, if it was a correct one, and

* "Elasticity and Fatigue of Wrought Iron and Steel," by Henry S. Prichard, *Industrial Engineering*, April, 1909, p. 15.

if the limits to elasticity could be noted and recorded, "with great ease," the engineer who failed to note the elastic limit in making tests or who omitted it from his published records of the tests would indeed be delinquent. Mr. Prichard.

This theoretical conception of ideal materials used only within the limits of perfect elasticity, however, does not harmonize with the facts as to the actual materials with which the engineer has to deal. Consider, for instance, the tests of eye-bars at the Watertown Arsenal* given in Table 5:

TABLE 5.—Compiled from Tests of Eye-Bars at the Watertown Arsenal.

Test No.	Material.	Normal size, in inches.	Load, in pounds per square inch.	Gauged length, in inches.	Permanent set, in inches.
4134	Steel.	5 × 1	5 000	260	Not recorded.
4135	"	5 × 1	5 000	260	0.0059
4136	"	5 × 1	5 000	260	0.0065
4137	"	5 × 1	5 000	260	0.0010
4138	"	5 × 1	5 000	260	0.
4139	"	5 × 1	5 000	260	0.0051
763	Wrought iron.	5 × 1⅛	5 000	180	0.0034
764	" "	5 × 1⅛	5 000	180	0.0006
765	" "	5 × 1⅛	5 000	180	0.0055
766	" "	5 × 1⅛	5 000	180	0.0031
767	" "	6 × 1½	5 000	180	0.0006
768	" "	6 × 1½	5 000	180	0.0003

In all the cases in Table 5 the first reading, after the micrometer was set at zero for the initial load of 1 000 lb. per sq. in., was at 5 000 lb. per sq. in.; thus the limit to perfect elasticity indicated for most of the bars was less than 5 000 lb. per sq. in. Recent tests by manufacturers exhibit similar results.

These permanent sets in eye-bars under low loads are small, and were appreciable only by reason of long gauged lengths and fine micrometric precision. In eye-bars and in smaller pieces of wrought iron and steel, when the direct load is increased slowly, a condition is eventually developed in which a great increase in elongation (or linear compression in crushing tests) occurs with comparatively little increase in load. The point at which this marked change in deformation occurs is properly termed the yield point, and is often called the elastic limit. It is well illustrated in a test of an eye-bar, made for the late George S. Morison, Past-President, Am. Soc. C. E., at the Watertown Arsenal,† and given in Table 6.

The yield point is difficult to observe accurately in making rapid tests of small pieces, and, usually, is not sharply marked in hard steel, in steel which has been worked cold, or in bending tests.

* Report for 1886, Part 2, pp. 1569–1617.

† Report for 1901, p. 410.

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TABLE 6.—Test of Eye-Bar for George S. Morison.

Gauged Length, 160 Inches.

Load, in pounds per square inch.	Elongation, in inches.	Permanent set, in inches.	Remarks.
1 000	0.0000	0.0000	Initial load.
5 000	0.0244	0.0005	
10 000	0.0529	0.0008	
20 000	0.1090	0.0017	
25 000	0.1375	0.0035	
28 000	0.1562	....	Called "Elastic Limit."
29 000	0.1653	....	
30 000	0.1830	0.0220	
31 000	0.2222	....	Yield point. (Term not used in report of test.)
32 000	{ 0.7800 0.8220	0.6461	Elongation 2 sec. later.
		Percentage of elongation	
33 000	1.03	0.64	
34 000	2.10	1.31	
35 000	2.97	1.86	
36 000	3.22	2.01	
37 000	3.44	2.15	
38 000	3.72	2.32	
39 000	3.98	2.49	
40 000	4.35	2.72	
41 000	4.67	2.92	
57 730	....	....	Tensile strength.
0	23.59	14.7	

Contraction of area, 53.5 per cent.

To illustrate the uncertainty as to the precise position of the yield point in bending tests of I-beams, the U. S. Board's Tables I, II, IV, and XI of tests of wrought-iron I-beams are here reproduced.

These tests were made under the direction of, and are given in a report signed by, the noted civil engineers, William Sooy Smith, M. Am. Soc. C. E., and the late Q. A. Gillmore, M. Am. Soc. C. E., Lt.-Col. of Engineers, Brevet Major-General, U. S. A.* The Report states, among other things:

"The loads were increased in each case until unmistakable signs of the failure of the beam appeared.

"The indications which have been relied upon for determining the elastic limit are as follows:

"1st. An unusual increase in the increments of deflection per 1 000 pounds and a corresponding decrease in E.

"2d. The set becoming appreciable and beginning to increase rapidly.

"3d. Often by a point of contrary flexure in the deflection curve, which becomes particularly noticeable when the curve of difference (G-S) is platted.

"Lastly, a general inspection of the diagram itself."

* Report of U. S. Board on Testing Materials, 1881, Vol. 2, p. 215.

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U. S. Board's Table I, 1881, Vol. 2, Page 226.

15-in. Beam. Clear span, 20 ft. Length, 20 ft. $10\frac{1}{8}$ in. Total weight, 1 012.5 lb. Weight per yard, 145.73 lb. Moment of Inertia, 536.56.

Number of trial.	Weight applied at center, in pounds.	Deflection, in inches.	Deflection, in inches per 1 000 lb.	Permanent set, in inches.	Permanent set, in inches per 1 000 lb.	Lateral deflection.	Coefficient of elasticity, *E*.
1........	9 306	0.185	0.0187	0.00	0.00	....	38 310 000
2........	12 237	0.24	0.0187	0.00	0.00	....	38 275 000
3........	15 566	0.33	0.0204	0.015	0.0009	....	35 021 000
4........	17 722	0.395	0.0216	0.035	0.0017	....	33 160 000
5........	19 586	0.46	0.0227	0.045	0.0022	....	31 362 000
6........	21 225	0.525	0.0241	0.055	0.0025	....	29 843 000
7........	22 351	0.56	0.0243	0.095	0.0041	....	29 376 000

Remarks.—The limit of elasticity does not appear to have been reached in this test, so far as can be discovered from the columns of sets and deflections, or from the diagram, unless, indeed, it is reached in No. 7.

The unit strain, f, for No. 7 is 22 400 lb., which appears to be too low for an elastic limit with a beam which has so high a modulus of elasticity.

$$E.\ L. = 22\,351 \text{ plus } 506.$$
$$f = 22\,400.$$
$$E_m = 33\,621\,000.$$

U. S. Board's Table II, 1881, Vol. 2, Page 228.

$10\frac{1}{2}$-in. I-Beam. Clear span, 22 ft. Length, 23 ft. $8\frac{1}{2}$ in. Total weight, 1 033 lb. Weight per yard, 130.68 lb. Moment of Inertia, 221.86.

Number of trial.	Weight applied at center, in pounds.	Deflection, in inches.	Deflection, in inches per 1 000 lb.	Permanent set, in inches.	Permanent set, in inches per 1 000 lb.	Lateral deflection.	Coefficient of elasticity, *E*.
1........	6 343	0.445	0.0641	0.0	0.0	0.0	26 960 700
2........	7 010	0.490	0.0645	0.0	0.0	0.0	26 836 900
3........	7 728	0.510	0.0614	0.0	0.0	0.0	28 217 200
4........	8 362	0.590	0.0654	0.0	0.0	0.0	26 264 900
5........	8 827	0.610	0.0649	0.0	0.0	0.0	26 704 700
6........	9 285	0.630	0.0638	0.0	0.0	0.0	27 113 200
6........	9 285	0.650	......	0.03	0.0031	0.0	....
7........	9 683	0.670	0.0650	0.03	0.0023	0.0	26 520 900
8........	10 138	0.710	0·0663	0.03	0.0023	0.0	26 134 100
9........	10 541	0.740	0.0664	0.03	0.0027	0.0	26 015 700
10........	10 940	0.770	0.0667	0.04	0.0034	0.0	25 897 500
11........	11 476	0.810	0.0670	0.05	0.0041	0.0	25 762 000
12........	12 216	0.845	0.0660	0.050	0.0040	0.0	26 208 200
13........	12 895	0.905	0.0670	0.065	0.0048	0.0	25 767 200
14........	13 831	0.960	0.0667	0.065	0.0045	0.0	25 975 600
15........	14 611	1.010	0.0664	0.065	0.0042	0.0	26 024 400
16........	15 543	1.085	0.0674	0.065	0.0040	0.0	25 709 800
17........	16 675	1.195	0.0690	0.070	0.0041	0.0	24 981 400
18........	17 701	1.280	0.0700	.....	......	0.0	24 706 300
18........	17 701	1.320	......	0.105	0.0056	0.0	....
19........	18 647	1.375	0.0715	0.120	0.0063	0.0	24 188 200
20........	19 565	1.505	0.0746	0.260	0.0130	½ in.	23 152 900
21........	20 236	1.645	0.0791	0.320	0.0150	½ in.	21 887 300
22........	20 725	1.770	0.0831	0.370	0.0176	¾ in.	20 819 000
23..	21 204	.....	......	.....	......	...	....
24........	22 436	2.465	0.107	1.04	0.0450	...	16 148 600

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Remarks.—The beam could not be broken vertically, as it yielded by buckling laterally.

Trial No. 6 was continued thirty-nine hours.

Trial No. 24 was repeated, the second application of the load causing the beam to bend sidewise.

The elastic limit appears, both from the table and from the curve diagram, to have been reached at Trial No. 19 with a load of 18 647 lb., as here the columns of reduced deflections and sets show the first decided irregularities and abnormal increase. The diagrams show this still more clearly.

To the applied load, 18 647 lb., must be added half the weight of the beam between the bearings, *viz.,* 477 lb. for the total concentrated load, namely, 19 124 lb. This equals a distributed load of 38 248 lb.

$$E.\ L. = 18\,647 \text{ plus } 477.$$
$$f = 29\,866.$$
$$E_m = 26\,099\,400.$$

The deflections from additional loads do not seem to be affected by the previous repeated loads, although left on for some time.

U. S. BOARD'S TABLE IV, 1881, VOL. 2, PAGE 232.

10½-in. Beam. Clear span, 22 ft. Length, 29 ft. 3 in. Total weight, 1 030 lb. Weight per yard, 105.63 lb. Moment of Inertia, 174.75.

Number of trial.	Weight applied at center, in pounds.	Deflection, in inches.	Deflection, in inches per 1 000 lb.	Permanent set, in inches.	Permanent set, in inches per 1 000 lb.	Lateral deflection.	Coefficient of elasticity, *E*.
1........	4 325	0.325	0.0677	0.00	0.0	....	32 587 000
2........	6 301	0.475	0.0701	0.00	0.0	....	31 472 000
3........	6 968	0.525	0.0705	0.00	0.0	....	31 220 000
4........	7 686	0.585	0.0716	0.005	0.0005	....	30 702 000
5........	8 320	0.635	0.0722	0.005	0.0005	....	30 504 000
6........	8 785	0.675	0.0728	0.015	0.0016	....	30 176 000
7........	9 243	0.715	0.0735	0.015	0.0015	....	29 229 000
8........	9 641	0.735	0.0728	0.015	0.0014	....	30 337 000
9........	10 096	0.765	0.0722	0.015	0.0014	....	30 425 000
10........	10 499	0.800	0.0727	0.015	0.0013	....	30 202 000
11........	10 898	0.835	0.0732	0.015	0.0013	....	29 986 000
12........	11 434	0.89	0.0745	0.025	0.0021	....	20 462 000
12........	11 434	0.90	....	0.045	0.0037	....	....
13........	12 174	0.96	0.0761	0.045	0.0035	....	29 013 000
14........	12 853	1.00	0.0752	0.045	0.0033	....	29 341 000
15........	13 798	1.07	0.0748	0.045	0.0031	....	29 376 000
16........	14 578	1.17	0.0780	0.075	0.0050	....	28 270 000
17........	15 501	1.31	0.0820	0.115	0.0071	....	26 836 000
17........	15 501	1.32	....	0.155	0.0097	....	....

Remarks.—Elastic limit at No. 15:

$$E.\ L. = 13\,793 \text{ plus } 397.$$
$$f = 28\,221.$$
$$E_m = 30\,270\,100.$$

Average of Tests 2 and 3, beams same size and span: Mr. Prichard.

$$E = 29\,409\,000.$$
$$f = 27\,159 \text{ lb.}$$

Average load at elastic limit = 13 270 plus 390.

U. S. BOARD'S TABLE XI, 1881, VOL. 2, PAGE 244.

8-in. Beam. Clear span, 14 ft. Length, 16 ft. 5¾ in. Total weight, 353 lb. Weight per yard, 64.29 lb. Moment of Inertia, 62.34.

Number of trial.	Weight applied at center, in pounds.	Deflection, in inches.	Deflection, in inches per 1 000 lb.	Permanent set, in inches.	Permanent set, in inches per 1 000 lb.	Lateral deflection.	Coefficient of elasticity, E.
1........	4 189	0.255	0.058	0.005	0.001	....	27 199 000
2........	5 351	0.335	0.060	0.005	0.0009	....	26 196 000
3........	6 364	0.380	0.058	0.015	0.0023	....	27 420 000
4........	7 749	0.460	0.058	0.015	0.0019	....	27 275 000
5........	8 848	0.535	0.059	0.020	0.0022	....	26 773 000
6........	10 159	0.615	0.059	0.030	0.0029	....	26 663 000
7........	10 961	0.660	0.059	0.045	0.0040	....	26 773 000
8........	12 237	0.740	0.059	0.070	0.0056	....	26 611 000
9........	13 851	0.930	0.066	0.155	0.011	....	23 915 000

Remarks.—The limit of elasticity is not so clear in the 8-in. beam as could be desired, and its determination is largely a matter of judgment.

Limit of elasticity assumed at No. 8.

$$E.\ L. = 12\,237 \text{ plus } 165.$$
$$f = 33\,957.$$
$$E_m = 26\,863\,750.$$

"It would seem as if too little attention has heretofore been given to *set,* as the data referring to it are very meagre as compared with that on deflection."

In only seven of the twenty-six beams tested by the U. S. Board is the elastic limit stated without qualification. It is evident that what they endeavored to determine was not the limit to perfect elasticity but a point corresponding as nearly as might be to the yield point in tension tests, that it was not well marked, and that it was nominally established somewhat arbitrarily.

To revert to the manufacturers' tests of I-beams epitomized in Table 4: Even if it had been possible to obtain, from diagrams given in the report of these tests, the points at which the first minute permanent sets occurred, it would not have been proper to have published these points as "the elastic limit," in view of the prevalent use of this term to indicate the yield point, any more than it would have been proper for the Watertown Arsenal to have indicated the

Mr. Prichard. elastic limit of the eye-bar for Mr. Morison (Table 6) as 5 000 lb. or less.

The writer could have obtained and published so-called elastic limits by a general inspection of the diagrams and by noting where the permanent set began to increase rapidly, as was done by Smith and Gillmore, and, had these limits been thus obtained and published, the average of the elastic limits thus published for the standard I-beams would have been in excess of the average for the new shapes by a percentage not far from, but somewhat greater than, the 18.6% by which the loads required to produce 0.1 in. of permanent set in standard beams exceeded those required to produce the same set in the new shapes. Such a determination, however, would of necessity have been in most cases arbitrary and subject to whatever influence the writer's prejudice, if he had any, might have exerted. For these reasons the elastic limits were not published.

It is explained in the paper that the seemingly low limits to elasticity observed in many experiments are due to internal stresses; and the investigations of Thomson, Thurston, and Bauschinger are referred to as showing this fact. The real elastic limit of the material in such cases is the computed stress plus the initial internal stress. It is further pointed out in the paper that the investigations of the authorities cited indicate that overstrained iron and steel recover their elasticity after a rest, that the yield point is raised by loading, and that the elasticity is perfected up to the amount of the load within limits somewhat in excess of the original yield point.

Those who hold with Mr. Worthington that there is a well-marked, easily-determined limit to perfect elasticity and that "just as soon as the extreme fibers of a beam are stressed beyond the elastic limit of the material, the beam has failed and is no longer useful in the art of construction," would do well to familiarize themselves with swaging, cold-rolling, wire-drawing, and rod-twisting, none of which would be possible if overstraining permanently destroyed the elasticity of the material. They should also consider the effects of punching, plate and shape straightening, and other shop operations which would weaken the material to such an extent as to destroy its usefulness, if it could not recover. They should then consider the internal stresses developed in the cooling of castings, beams, etc.

A good illustration of the raising of the yield point and the perfecting of elasticity is afforded by a test of an eye-bar at the Watertown Arsenal, as condensed in Table 7.

In eye-bars, the great elongation at their yield points destroys their usefulness as members of a structure after this point is reached; in compression members, unless they are very short and solid or compact, failure will take place from buckling or local crippling at the yield point or before it is reached; but it appears from the investigations

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TABLE 7.—TEST NO. 4136 OF STEEL EYE-BAR (NOMINAL SIZE, 5 BY 1 IN.) AT THE WATERTOWN ARSENAL, REPORT FOR 1886, P. 1578.

Load, in pounds per square inch.	In 260 Inches. Elongation, in inches.	In 260 Inches. Set, in inches.	Elongation from center to center of pins (25 ft. 8 in.), in inches.	Remarks.
1 000	0.	0.	0.	Initial load.
5 000	0.0456	0.0065		
10 000	0.0915	0.0085		
15 000	0.1369	0.0089		
20 000	0.1815	0.0096		
25 000	0.2264	0.0101		
30 000	0.2720	0.0109		
.....	......	......		
37 330	0.3459	......	......	Called "Elastic Limit."
37 710	0.3700	......	......	
38 100	0.5665	......	......	(Yield point.)*
39 000	1.07			
40 000	2.35	......	2.76	
41 000	......	......	3.30	
42 000	......	......	3.61	
43 000	......	......	4.02	
56 000	......	......	10.70	
......	.....	......	.....	
64 000	......	......	18.35	
65 000	......	......	19.83	
65 750	......	......	21.05	
0	......	......	.....	Rested 5 minutes.
66 000	......	......	21.35	
70 286	......	30.42	36.60	Maximum load reached.
	In 290.42 Inches.			
	Elongation.	Set.	......	Rest—Duration not stated.
1 117	0.	0.	......	Micrometer reset.
39 095	0.4236			
55 850	0.6310	— 0.0010	......	Note the minus sign.
2 013	.....	......	......	Rested 1 hour.
1 117	......	— 0.0150		
1 117	0.	0.	.....	Micrometer reset.
55 850	0.6263	0.0028	.. ...	Immediate set.
		0.0005	......	Set after 10 minutes.
		0.0000	......	Set after 12 minutes.
1 117	0.	0.		
16 755	0.1685	.....	......	Rested 15 minutes under load.
72 605	0.8660	......	......	(Yield point.)*
72 605	0.8750	0.0301	......	Immediate set.
		0.0215	......	Set after 20 minutes.

* The term "yield point" does not appear in the report of the test.

and tests cited that it is reasonable to expect that I-beams, when supported laterally, can have their elasticity perfected and their permanent strength enhanced, without undue deformation, by overstraining, unless they have been rolled so thin that their resistance to local crippling or other influence of attenuation is the determining factor.

It is not well, however, to assume a permanent increase, in the strength of the material of which I-beams are composed, much beyond its original yield point, as Bauschinger's experiments have shown, for

Mr. Prichard. material so strained that there is, as time goes on, a gradual yielding; besides, there is the danger of an insidious fatigue by the gradual extension, in the hardened material, under intense stresses, of microscopic flaws into planes of rupture, with little warning in the way of deformation.

If the elasticity of steel can be perfected up to the yield point by overstraining, and if the material has not been rolled too thin, I-beams, according to the theory explained by Professor Thomson and adopted by his brother, Lord Kelvin, in his article on elasticity,* can have their permanent capacity in one direction raised above that computed by the ordinary theory of flexure without straining the material appreciably beyond its original yield point, and without undue permanent set. Its elasticity under a load in the opposite direction, however, according to this theory, would be reduced.

If one of the 15-in. standard I-beams, tested as described in the paper, was loaded until the horizontal layer of material half way between the neutral axis and the extreme fiber was strained to the yield point, the material between this layer and the extreme fiber would be strained very little above the yield point, and the resistance of the beam at this stage of the test would, in consequence, be about 15% greater than it was when the extreme fiber first reached the yield point.† If this critical load should now be removed and the beam be allowed to rest, or even if it rested with the load on, it would thereafter, according to Thomson's theory, withstand any subsequent application of the load in the same direction without any additional permanent set.

As the yield point of the flanges of these standard 15-in. beams was a little more than 38 000 lb. per sq. in., and the working load is based on 16 000 lb. per sq. in., the permanent capacity thus indicated is about $2\frac{3}{4}$ times the working load $\left(W \frac{38\ 000}{16\ 000} \times 1.15\%\right)$. For the new shapes of 15-in. beams tested, using the higher yield point shown in the specimen tests of the flanges (40 000 lb.), the result is the same, $2\frac{3}{4}$ times the working load. This theory, however, assumes that the metal is sufficiently compact practically to avoid the weakening influence of attenuation, an influence difficult to analyze and only satisfactorily determinable by experimental investigation.

The permanent set can also be indicated by the theory of overstrained beams, but not, even relatively, by any mere comparison of flange widths. Instead, it involves the consideration of stress and

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* Encyclopedia Britannica, 9th Edition, p. 798.

† The yield points in tension and compression in steel are about the same, and are here so assumed. A layer quite close to the neutral axis could be assumed as the one strained to the yield point and it would make very little increase in the computed result, but the assumption that the material between this layer and the extreme fiber would be strained very little above the yield point would not then be correct.

strain in each element of the beam throughout its entire length, and is a very complicated problem. Mr. Prichard.

The time required to regain elasticity after overstraining is a subject of inquiry by Mr. Belzner. It probably depends on the size of the piece and the severity of the overstrain. At first the recovery is rapid, then it goes on gradually for hours and, possibly, for days. Overstraining wrought iron and steel makes the metal partly plastic. The viscosity of the plastic portion retards the tendency of the strictly solid portion to regain its shape. The hardening of the plastic portion is accompanied by an increase in tensile strength. Until the strength ceases to increase, the plastic portion has not completely hardened. The U. S. Board investigated the increase in tensile strength of overstrained wrought iron, with the following results:*

Average gain in	less than 1 hour	1.1%	(5 tests)
	less than 8 hours and more than 1 hour	3.8%	(8 tests)
	1 day	8.9%	(5 tests)
	3 days	16.2%	(10 tests)
	8 days	17.8%	(2 tests)
	6 months	17.9%	(12 tests)

The U. S. Board's Table II shows that Trial No. 6 was continued 39 hours. The writer knows of only one other endurance test. It was made by a U. S. Government engineer on a pair of 15-in. light iron I-beams (50 lb. per ft.) some years before 1884, by applying a uniformly distributed load on a 21-ft. clear span; the account† is as follows:

"They showed no signs of breaking with the maximum load applied, but could not be loaded further as they had deflected so that one of them touched the ground. In testing these beams, a load equal to twice the safe load‡ was first applied and allowed to remain on the beam 23 days; during this time the deflection increased from 0.98 inches to 1.12 inches. The load was then increased up to three times the safe load, and allowed to remain 15 hours, in which time the deflection increased from 2.01 to 2.09 in.; the load was then increased to 90 000 lb. [3.6 times the safe load] with a deflection of 2.7 in., which, after 18 hours, had increased to 2.77 in. The load was allowed to remain on the beams 15 days, when it was removed."

Referring to Mr. Vilar y Boy's discussion: The introduction of empirical formulas for flexure, based on breaking loads and used with a large factor of safety, marked an advance in engineering; and the high regard for experiments by the men who introduced these formulas is an example for modern engineers; but it is the permanent strength, rather than the temporary strength shown by the immediate breaking

* Report of U. S. Board for Testing Materials, 1881, Vol. 1, pp. 107-111.

† As given in the New Jersey Steel and Iron Company's "Book of Useful Information."

‡ Based on 12 000 lb. per sq. in., extreme fiber stress.

Mr. Prichard. load, that should be investigated; and, as far as practicable, correct theory should be used in formulating the results of these investigations.

Referring to Mr. Dunham's discussion: In general, for the same quality of steel, when one section is made smaller than another by greater reduction in rolling, it should, theoretically, and does, usually, have a higher yield point and greater ultimate strength, and this fact should be properly considered in comparing the effects of overstraining in different sections.

The theory of overstrained beams applies to all symmetrical sections in which the metal is not too attenuated, and, therefore, should apply to rails. When compact symmetrical rolled beams, including I and rail sections, are moderately straightened by forces applied in the same direction and with the same distribution as the loads to which the beams will subsequently be subjected, theory indicates that their elasticity in the direction of the loads will be improved and its limit raised. If the amounts which the beams have to be straightened are great, or if the straightening forces and subsequent loads differ more or less in direction or distribution, the problem is complicated, and the great impact to which rails are subjected in service introduces further complication, as do other elements.

The recuperative and adaptive power of steel is a real reliance against the effects of the necessary operations of the mill and shop after the material leaves the rolls, but it is too much to expect that the net result will always be a gain in strength. The products of the mill and structural shop should be used with a reasonable margin of safety.

Theory is useful in suggesting lines for experimental investigation, in interpreting results of experiments, and in giving to these results their widest application, but intelligent observation and experiments should take precedence; they are what Mr. Dunham terms "real evidence."

Referring to Mr. Williams' discussion: Mr. Williams has made an analysis of the errors involved in the assumption that originally plane cross-sections remain plane during flexure, which is superior to that given in Section 1 of the paper, because it is general, instead of being restricted to special cases, and because it is an accurate instead of an approximate deduction from the assumptions common to both analyses.

In deriving shear distortion, these assumptions involve the use of the ordinary theory, which makes the computed error in bending stress too large by the amount of a secondary error. When the computed error is not very large, the secondary error is so small that it is negligible. Its neglect, however, when the computed error is very large, makes the latter excessive and only roughly approximate.

The results of Mr. Williams' analysis are not directly comparable with those given in Table 1 for two reasons: First, his percentages of error are based on the extreme fiber stresses indicated by the ordinary theory, while those in Table 1 are based on the capacities indicated by that theory, and, therefore, are lower, 37% error in capacity corresponding with 58.7% error in extreme fiber stress; 11% with 12.4%; 1.24% with 1.25%, etc. Second, his percentages of error vary properly throughout the length of each girder, while those given in Table 1 are, in each case, the mean of these various percentages. Of the percentages of error at the centers of the girders, given in Table 1, each is one-third less than the mean. Mr. Prichard.

After allowing for these differences, the comparison of the computed errors at the centers of the uniformly loaded girders, 4A, 4D, and 4E, for the condition that the length equals ten times the depth, is as follows:

	Girders.		
	4A.	4D.	4E.
By Mr. Williams' method..	0.84%	0.38%	0.35%
By the writer's method......	0.84%	0.37%	0.34%

This close accord is very satisfactory. In addition, however, the writer checked all Mr. Williams' analyses except the omitted analysis by which the equations (34 to 38, inclusive) for plate girders were developed, and all the applications to specific cases.* To have developed Equations 34 to 38, inclusive, independently, would have delayed the closing discussion and did not seem worth while, as they agree, when applied to I- and rectangular beams, with the checked equations for such beams; and when applied to the specific plate girders, 4A and 4D, the results agree as well as could be reasonably expected with those of the writer's analysis of these girders, adjusted for comparison, as previously shown.

Mr. Williams' analysis shows conclusively that the ordinary theory of flexure is very faulty, when applied to very short girders; but it does not give for such girders a substitute for or an accurate correction of the said theory. For such girders, as he has pointed out, a consideration of the secondary errors (which both Mr. Williams and the writer have neglected) and of the effects of the manner of applying the loads and reactions with respect to the vertical as well as to the horizontal distribution of points of application, are essential to even a fairly accurate determination of the stresses within the elastic limit.

An accurate or close determination of the stresses in a very short girder is such an intricate problem that it seems wholly impracticable

*Greater precision in applying Mr. Williams' equations indicates a percentage of error for Girder 4D slightly smaller than that given in his discussion.

Mr. Prichard. to make an analysis. There is little doubt, however, that the elastic limit in the extreme fiber of such girders is reached under much smaller loads than those indicated by the ordinary theory of flexure.

In view of this disconcerting fault in the ordinary theory in indicating excessive capacity within the elastic limit, it is comforting to know that there are good reasons for believing, as explained in Section 6 of the paper, that, after the elastic limit is reached, a high permanent resistance can be developed in short girders without enough permanent set to be serious, provided the metal is not spread out too thinly and the loads are not reversed. In the case of pins and square sections, experience and tests justify this belief, but for I-beams and plate girders repeated and endurance tests are needed.

From a practical standpoint, Mr. Williams' analysis is especially interesting with respect to concentrated loads on girders of the lengths ordinarily used in practice. It is not reasonable to suppose that concentrated loads can actually be concentrated at a point, as is generally assumed for convenience in making analyses and computations, and his Equations 47 to 51, inclusive, show this by indicating an infinite percentage of error for the condition, a (the half length of distribution of load) equals zero. Loads have to be somewhat distributed, and the length of this distribution is, as Mr. Williams shows, an element in determining the stresses.

He has made his analysis of concentrated loads for the condition of uniform distribution over a distance, $2a$, at the center of the girder, and by so doing has developed the interesting Equations 47, for rectangular beams, 48 and 49 for I-beams, and 50 and 51 for plate girders. The method by which these equations were developed can readily be applied to other conditions of concentrated loading and to other beams and girders. His analysis, as given, shows that, even for girders of ordinary length, the error in the ordinary theory may be considerable, if the web is thin and the load concentrated within a short distance. For Girder 4A, which has 14 sq. in. in each flange, a 40 by $\frac{1}{4}$-in. web, and, for the case considered, a length of 420 in., with a load uniformly distributed over 12 in. at the center, the stresses are shown to be 14% greater than those indicated by the ordinary theory of flexure. The excess in the stress, in such cases, is confined to a short distance at the center, and results, as can be shown theoretically, in a fairly sharp bend at the center in deflecting, a condition which, the writer has been informed, is quite apparent in tests of plate girders.

Referring to Mr. Godfrey's discussion: The absence of lateral support from the beams tested by Professor Marburg would not necessarily affect to any considerable extent the behavior of the beams in the early stages of the tests. If it had had any considerable effect before the elastic limits recorded by Professor Marburg were reached, the fact would have been evident in the deformation of the beams; but, in the case of the beam with the low elastic limit of 10 800 lb. per sq. in., cited by the writer, the account of the tests expressly states, "the flanges remained in perfect alignment until shortly before failure." Mr. Prichard.

The account of the tests shows that the manner of loading was eventually reflected in the way in which the beams failed. Under ideal conditions both the columns and the compression flanges of beams may be on the verge of failure from insufficient stiffness, without giving any evidence of the fact in the way of deformation or increased stress intensity.

Referring to Professor Marburg's discussion: The statement in the paper regarding certain manufacturers' tests of I-beams is not a report, but simply a brief epitome by the writer of portions of an unpublished report (with which he had no connection) of tests he had no part in making. This unpublished report, by the engineers who conducted the tests, to the concerns for which they were made, not having been presented to the Society, is not open for discussion.

Naturally, the epitome, the essence of which is concentration of results and absence of elaborate details, differs greatly from Professor Marburg's conception of what a full and minute report should be. By regarding the epitome as a report he is led to criticize the very points which constitute its essential nature, and to raise the question whether everything not recorded in the paper was omitted from the records of the tests. Undoubtedly, these records cover an extensive range of investigation and contain a multiplicity of detail, most of which would naturally be excluded from an epitome.

The writer fully sympathizes with Professor Marburg's desire for information regarding these tests. This desire is doubtless general throughout the Profession. It was for this reason, and because of its appropriateness to a paper which criticized manufacturers of I-beams for having relied solely on theory, that the writer made the epitome, sought permission to include it in his paper, and suggested to those who conducted the tests that, by presenting full details to the Society, they could make a valuable contribution to engineering knowledge.

If a detailed account of this series of tests is presented to the Society, it should be done, preferably, by the engineers who conducted them. It was not possible for the writer to include such an account when he presented his paper, and even if it had been, and if he had chosen to do so, he would have included only the epitome, and would have presented the detailed account of the tests in a separate paper under an appropriate title. It does not rest with the writer, but with others, to decide whether or not such a paper shall be presented.

Mr. Prichard. This series of tests, which included, in addition to the I-beams referred to in the paper, many I-beams of other sections, as well as plate girders, beam connections, etc., was undertaken by the manufacturers cited in the writer's reference to Mr. Stern's discussion, not only to determine by actual trial the strength of plate girders, beam connections, I-beams, etc., of existing designs, instead of depending solely on theory, but also to obtain information which would be useful in making new designs.

The Carnegie Steel Company has applied the information thus obtained in designing new beams, of lighter models than those adopted by the Association of American Steel Manufacturers in 1896. These beams are adapted to conditions which frequently arise in buildings where standard beams of the depths required by considerations of stiffness or types of floor construction have an excess strength. These supplemental sections have been introduced since the writer's paper was presented to the Society.* The notable features in their design are that the flanges are not unduly wide in proportion to their thickness, and the connections between the flanges and the webs have ample fillets of special design giving wide roots. The webs are thinner than in the corresponding standard beams, and when shear or heavy concentrated loading is the governing consideration this fact should be properly considered. This caution has already been emphasized in Section 2 of the paper.

Engineers will applaud the spirit which prompts manufacturers to make, through their engineering staff, searching investigations and experiments as a basis for new designs; but when it comes to publishing these investigations there are some who will object and claim that the facts should be brought out "under wholly disinterested auspices."

In general, the specific points raised by Professor Marburg have been discussed in replying to Mr. Worthington's discussion, but there are a few which call for separate consideration.

The conditions of loading are outlined on page 917. These conditions and the connection angles cited in the outline were those which are usual in ordinary practice.

While the epitome of the tests does not refer to the roots, the material in the roots was tested, its behavior during the progress of the I-beam tests was carefully observed, and the conclusions drawn therefrom are reflected in the roots of the supplemental I-beams designed since the tests were made.

Professor Marburg points out that deflections were observed, and he then makes the following inquiry and statement:

"Why, then, are the elastic-limit loads not reported? The 'elastic-limit load' may perhaps most reasonably be defined as that load

* The sections referred to are described and their elements given in a pamphlet issued by the Carnegie Steel Company.

above which the load-deflection ratio ceases to remain constant. In the writer's [Marburg's] tests this load was usually found to be greater, and in some cases much greater, than the load at which permanent set (0.01 in.) was first observed." Mr. Prichard.

To have recorded the elastic limit in accordance with Professor Marburg's conception would have corresponded neither with the usual idea of it as the point at which a sudden and considerable increase in the rate of yield begins, nor with a strict definition which has been well worded in the Standard Dictionary, as follows:

"The limit of elasticity is the point of stress beyond which an elastic body loses power to return completely to its former shape and size."

In bending tests the yield point is not usually well marked, and the strict limit of elasticity of steel as it comes from the rolls cannot, with the present approved means of measuring, be certainly known. In general, for the same quality of steel and within the limits of experience, the greater the gauged distance and the more precise the micrometer the lower the point below which the limit of elasticity is shown to be. In many cases of eye-bars and columns, it is less than 5 000 lb. per sq. in. If no change took place in the quality of the steel when strained beyond this amount, thousands of eye-bars which daily are so strained many times would gradually be elongated by the accumulation of successive permanent sets until failure of the bars or the structures of which they are a part would surely follow. Such being the case, the writer is compelled to regard as an improvement the change in quality whereby the accumulation of permanent sets is arrested.

Professor Marburg states that he does not know of any experimental evidence which can be looked to for a conclusive answer to the question "whether extreme fiber stresses in excess of the elastic limit are to be regarded as necessarily dangerous under conditions in which permanent set *per se* is not objectionable." In view of the fact that the limit of perfect elasticity in many cases can be shown, if measured with great precision, to be very low, it is to be hoped that those who make I-beam tests in the future will seek for such experimental evidence.

The writer gave the magnitudes of the loads which caused permanent sets of 0.4 in. for the reason that it is a matter of interest to know that the beams under the condition outlined resisted gradually increasing loads up to and somewhat beyond these considerable magnitudes without buckling, twisting, or rupture, and without great change in shape. He did not have information as to the ultimate loads, but in no case was the full ultimate tensile strength of the beams developed by actual rupture.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

TRANSACTIONS

Paper No. 1235

AIR RESISTANCES TO TRAINS IN TUBE TUNNELS.*

By J. V. Davies, M. Am. Soc. C. E.

With Discussion by Messrs. George Gibbs, George H. Pegram, Charles S. Churchill, L. J. Le Conte, and J. V. Davies.

Introduction.

The development in recent years of tube tunnels and subways with close clearances, as integral parts in the construction of high-speed, electrically-operated railways for passenger transportation, has opened questions of somewhat serious moment in the operation of such lines, which had not previously been considered, although under these new conditions they have a material bearing on the operation, and should have some consideration in the design under which such tunnels are constructed.

An interesting question of this character, regarding which, as far as the writer can find, little information has been published or is available, is that of the resistance of air and the movement of columns of air in the operation of trains in tube tunnels. There has been available a considerable mass of information as to the resistance of air and pressures on trains operating in the open, but it will be obvious that such information has comparatively little, if any, bearing on this question when trains are operated in a confined aperture such as a tube tunnel, where the relation of the cross-section of the aperture to the cross-section of the moving train introduces features which in no

* Presented at the meeting of May 15th, 1912.

possible way relate to the same pressures and the same conditions as when the train is operated in the open.

In the development of the Hudson and Manhattan Railroad the line has recently been extended in Jersey City to a connection on the surface of the ground with the tracks of the Pennsylvania Railroad near Prior Street, and a through service between New York City and Newark has been put in operation by electrically-operated trains of the tunnel type, running at high speeds. Consequently, there is introduced the operation of these same trains at varying speeds over the tracks of the Pennsylvania Railroad, across the Newark Meadows, and through the tube tunnels of the Hudson and Manhattan system to Church Street in the Borough of Manhattan. The question, therefore, of the power necessary to overcome these air resistances, is an important one.

In discussing the paper by George Gibbs, M. Am. Soc. C. E., on the New York tunnels of the Pennsylvania Railroad, the writer stated* that he had made some interesting experiments in relation to air resistances in tube tunnels, which at that time were not completed. Since then the substance of these experiments has been completed and is presented herewith. The results of the experimental tests of the conditions arising from the joint train operation, above mentioned, the writer presents, principally as a series of facts, with certain deductions drawn therefrom.

The formulas, given later, have either been deduced from the plotting of the actual results obtained in the experiments, or have been based on formulas elsewhere published for air resistances in the open air, introducing coefficients which have been obtained from the results of these experiments.

The subject matter of this paper is presented in the hope of starting discussion or drawing out information as to similar tests under corresponding conditions, which may be of assistance in the general proposition of ventilation and power consumption necessary in the operation of trains in tunnels constructed for transportation purposes, or from which may be obtained more definite information than is at present available, which may be of service to the Engineering Profession in the development of similar problems in the future.

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* *Transactions*, Am. Soc. C. E., Vol. LXIX, p. 414.

Test Runs.

On the morning of October 7th, 1911, tests were made to determine train resistances, with special reference to the resistance offered to train movement by the air, both in and out of the tunnels of the Hudson and Manhattan Railroad and on the surface tracks of the Pennsylvania Railroad between Jersey City and Manhattan Transfer. These tests were made in the early morning (from 1.20 A. M. to 5.00 A. M.) when there was practically no other traffic in the tubes. The test runs and their numbers are listed in Table 1; the graphical log for the first four is shown on Plate XIII.

TABLE 1.—Test Runs.

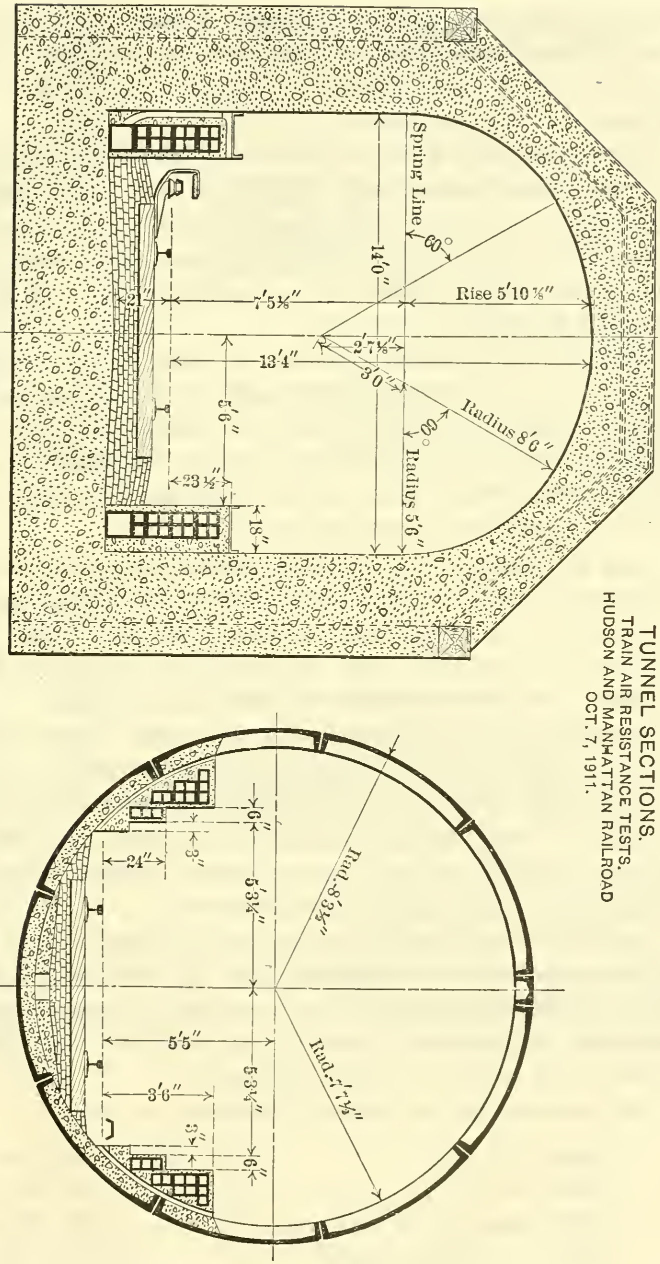
Run No.	From:	To:	Object of test.
1-*E*	Manhattan Trans	Church Street	Resistance.
2-*W*	Church Street	Manhattan Trans	"
3-*E*	Manhattan Trans	Church Street	"
4-*W*	Church Street	Manhattan Trans	"
V-5-*E*	Manhattan Trans	Church Street	Velocity.
V-6-*W*	Church Street	Manhattan Trans	"

Tests Nos. 1 to 4, called Resistance Tests, were made for resistance determination only; no air velocity readings were taken.

Tests Nos. 5 and 6, called Velocity Tests, were made for air velocities; that is, to determine the "slip" in the tunnels. By "slip" is meant that portion of the air which is not dispelled or given the same motion as the train, but either remains at rest or is deflected by the front of the train and passed back to the rear end.

Line.—The distance from Manhattan Transfer to Church Street Terminal is 40 454 ft., or 7.66 miles. Of this distance, 13 400 ft. is over the tracks of the Hudson and Manhattan Railroad, and is in tunnels (partly iron tubular and partly concrete lined); the remainder is over the tracks of the Pennsylvania Railroad, and is in the open.

On leaving Manhattan Transfer, the track passes over the Hackensack Meadows to Marion Station, a distance of about 4¼ miles. Throughout this distance the right of way is unprotected by cuts or buildings, and is practically on level grade, and with very slight curvature. From Marion Station to the Portal, 4 600 ft., the track passes through a rock cut, from 20 to 30 ft. deep, and wide enough for four



Concrete Tunnel.
Area 166 sq.ft.

Iron Tunnel.
Area 160 sq.ft.

TUNNEL SECTIONS.
TRAIN AIR RESISTANCE TESTS.
HUDSON AND MANHATTAN RAILROAD
OCT. 7, 1911.

Fig. 1.

tracks in some parts and for six or more in others. On entering the Portal the line passes through a single-track concrete tunnel (H. & M. standard, 14-ft. section, as shown by Figs. 1, 3, and 4) to the Pennsylvania Station (Jersey City).

There is no exit for the air, from the Portal to a point just west of Grove Street Station (4 000 ft.), but at that point, through an enlargement, the air can escape from one tube to another, or to the surface by way of the station passages. Between Grove Street and the Pennsylvania Station there are two enlargements, and the total distance is 2 800 ft.

Leaving the Pennsylvania Station, the tunnel is of standard iron construction (15 ft. 3 in. diameter inside of lining plate flanges, as shown by Figs. 1, 3, and 4), and this extends to Church Street (6 600 ft.) with no outlet for the air except at the Church Street end. The maximum rate of grade of the tunnel from the center of the river to Church Street is 4.63 per cent.

Area of Tunnels.—The net internal area of the concrete section of the tunnels is 166 sq. ft.; that of the iron section is 160 sq. ft.

Weather.—Rain was falling throughout the tests. Run No. 1-*E* was made in a very heavy rain; the others, with the exception of the last part of No. 4-*W*, were made in a more moderate rain.

Temperature.—The temperature in the tunnel was approximately 65° Fahr.; that in the open was approximately 55° Fahr.

Wind.—The wind was east-northeast, and was blowing at about 15 miles per hour. The direction of the wind was such that in the east-bound runs the train was running almost against the wind, while in the others it was running nearly before it.

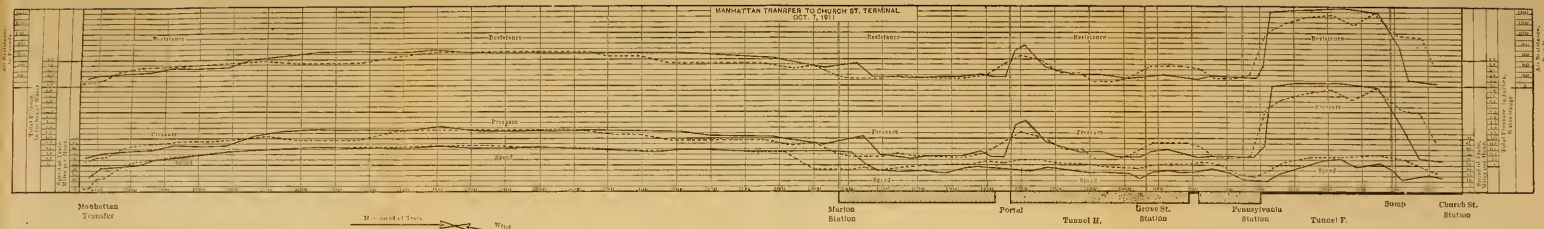
Make-Up of Test Train.—The test train consisted of three Hudson and Manhattan Railroad Company's Type "C," steel, motor cars. On account of the arrangement of the apparatus, the pressure runs had to be made with the train heading in the same direction. The total weight of cars and crew was about 106 tons.

The following are the principal dimensions of the cars:

Length over all	48 ft. 5 in.
Width over all	8 ft. $11\frac{1}{4}$ in.
Height above top of rail	11 ft. $8\frac{7}{16}$ in.
Center to center of trucks	33 ft. 0 in.

PLATE XIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXV, No. 1236.
DAVIES ON
AIR RESISTANCE IN TUNNELS.

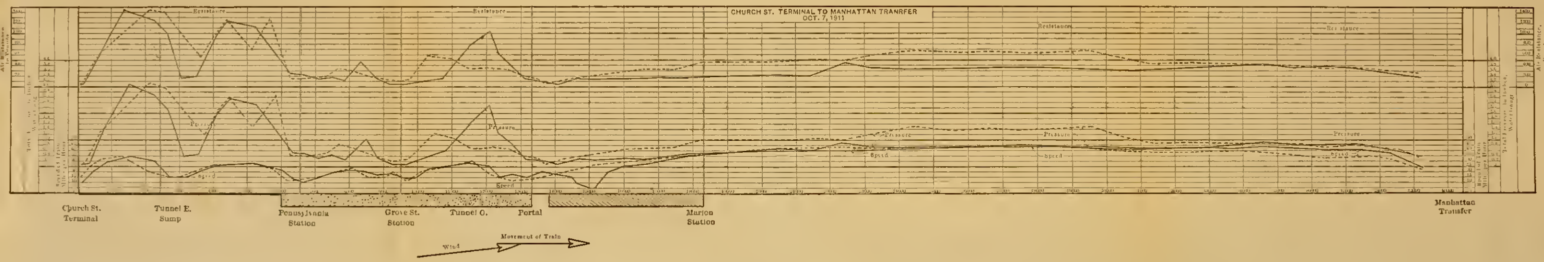
HUDSON & MANHATTAN R. R. CO.
TRAIN AIR RESISTANCE TEST.
GRAPHICAL LOG.



NOTE:-

Test No. 1 East.
Test No. 3 East.
Iron Section-Area 160f²
Concrete Section-Area 166f²
Rock Cut-Out of Tunnel
Out of Tunnel
Type C Car-90f²Area.

Wind-General direction E.N.E in the open, about opposed to direction of train movement, velocity about 10 to 15 m.p.h.
Weather-Rain
Temperature-In open 55° F. In tunnel 65° F.
Train-Three cars, total weight about 106 tons.
Cars-8'11¼" wide x 11'8½" high x 48'2" over buffers, Two motors each car 160 H.P. each.



NOTE:-

Test No. 2 West.
Test No. 4 West.
Iron Section-Area 160f²
Concrete Section-Area 166f²
Rock Cut-Out of Tunnel
Out of Tunnel
Type C Car-90f²Area.

Wind-General direction E.N.E. in the open, about the same as the direction of motion of train, 10 to 15 m.p.h.
Weather Rain
Temperature-In open 55° F.. In tunnel 65° F.
Train-Three cars, total weight about 106 tons.
Cars-8'11¼" wide x 11'8½" high x 48'2" over buffers. Two motors each car 160 H.P. each.

Wheel base.	Motor truck	6 ft. 6 in.
	Trailer truck	5 ft. 6 in.
Diameter of wheels.	Motor truck	34¼ in.
	Trailer truck	30 in.
No. of passengers.	Seated	44
	Total	150
Weight, in pounds.	On motor truck	41 730
	On trailer truck	27 890
	Total	69 620
Motors for each car.	Number	2
	Rating, each	160 h.p.
Cross-sectional area		90 sq. ft.

Arrangement of Apparatus.—The arrangement of the apparatus for both resistance and velocity tests is shown by Fig. 2.

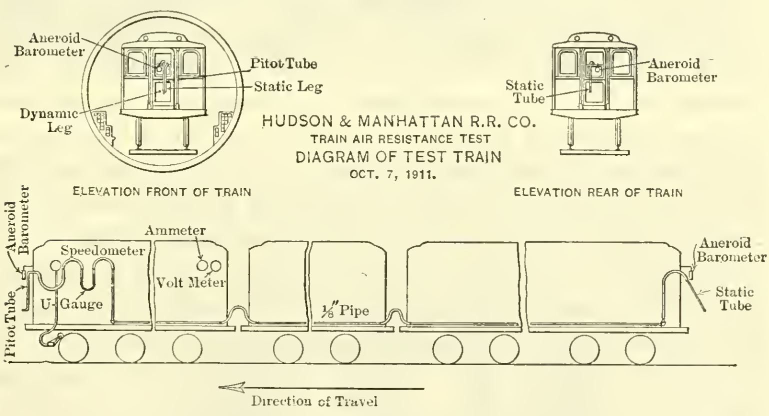


FIG. 2.

RESISTANCE TESTS.

Speed.—A Warner autometer, with railroad attachment, was used for the speed determination. The driving wheel of the mechanism was attached to the forward truck frame and was driven off the tread of the truck wheel. The flexible shaft from this wheel passed up through the floor of the car to the dial in the front vestibule.

Pressure.—A Pitot tube was held outside the window in the front door of the train, with the dynamic leg connected to one leg of a

U-tube, the opening in the dynamic leg looking in the direction of train movement. The other leg of the U-gauge was connected by a rubber hose to a $\frac{1}{8}$-in. pipe which extended the length of the train, the connections between the cars being made with rubber hose.

Outside the window in the rear door of the train a static tube was held, with its opening at right angles to the direction of motion of the train. This tube was of brass, having an inside diameter of $\frac{5}{32}$ in. with the end blanked. A very small hole (about $\frac{1}{128}$ in. in diameter) was drilled in the side of the tube. This static tube was connected to the $\frac{1}{8}$-in. pipe line, and the resultant reading of the U-gauge gave the total pressure, in inches, water gauge, the plus pressure in the front automatically adding itself to the minus pressure at the rear.

Outside the front and rear windows were also hung aneroid barometers, and the increase in pressure at the front was added to the decrease in pressure at the rear; the results, when the proper correction was made for elevation, gave the total pressure. Of course, the aneroid pressure should check with that found by the Pitot tube, and it was found that it did so, excepting that when the total pressure reached $1\frac{1}{2}$ in., water gauge, or greater, either the hose connections of the tubes, the pipe line itself, or the line connections between the cars, leaked, thus destroying the vacuum at the rear end. At all lower pressures the tube and aneroid readings checked very closely; in fact, in many instances, the readings were identical.

Horse-Power.—An ammeter and volt meter were set in the front car in order to measure the power input to the motors. This method gave the power consumption of the train, but, in calculating the results, it was very difficult to find just what part of this power was expended to overcome speed resistance and what part to overcome other resistances, such as those due to acceleration, grade, etc. The change in speed—acceleration or retardation—proved especially bothersome, as the train was found to change almost continuously.

Velocity Tests.

Each leg of the Pitot tube was connected to a leg of the U-gauge. The tube was held between the first and second cars of the train, with the opening of the dynamic leg looking in the direction of train motion, the end projecting beyond the car body.

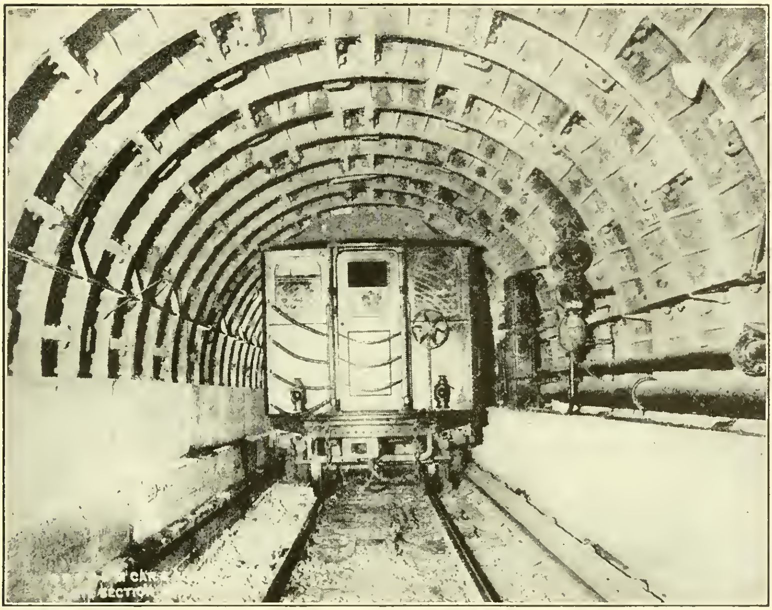


FIG. 3.—CAR IN IRON-LINED TUNNEL.

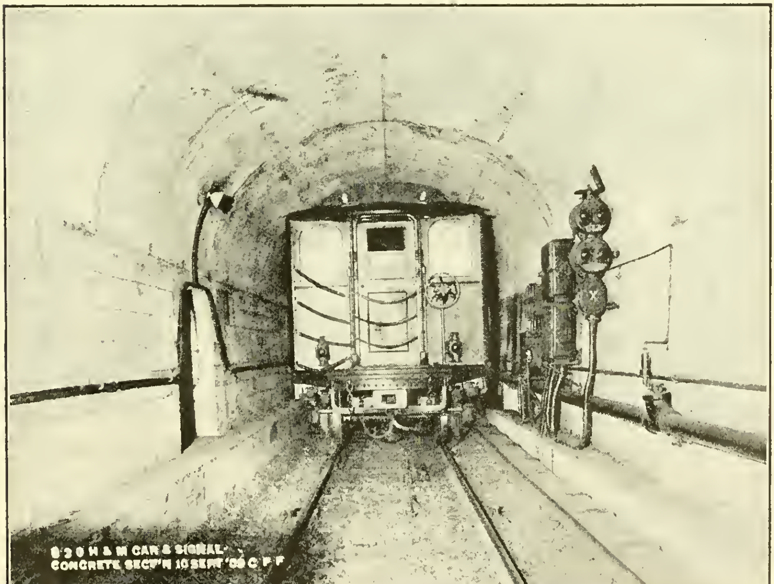


FIG. 4.—CAR IN CONCRETE-LINED TUNNEL.

Method of Tests.

Resistance Tests.—Two men were stationed at the rear of the train—one to read the rear aneroid and the other to record the results. Readings were commenced when the train left the starting point and were kept up continuously until the other terminal was reached. These readings were recorded for each interval of 15 sec., the number of readings per interval varying from 3 or 4 to 6 or 8.

One observer read the ammeter and volt meter and another recorded the readings, dividing them into intervals of 15 sec. The voltage was found to be quite constant for the different sections, depending on the distance between feeders and the pressures carried at the several sub-stations. The current consumption varied considerably, depending on the speed, grade, etc.

At the front end one observer read the speedometer, another the U-gauge, and a third the front aneroid. A fourth man kept the time divided into 15-sec. intervals, the same as in the other readings, and also gave the time of passing stations, entering tunnels, or other special points, so that the relation between the readings and the location of the train could be determined. Two men recorded these readings.

Velocity Tests.—In the velocity tests no readings were made of the resistance or power consumption. As mentioned before, the tube was held out of the opening between the cars, the position being changed from the right side, to the top, and to the left side.

As each leg of the Pitot tube was connected to the U-gauge, the resultant reading was the pressure due to the velocity only. The velocity corresponding to this pressure gave the gross air velocity, and from this should be deducted the speed that the tube itself was moving, the train speed, and this difference, or net velocity, was the velocity of the air passing back at the side of the train. The speed of this air, when multiplied by the area, gave the quantity passing back, or the "slip." Care was taken to have the tube project as far from the car as possible, for it was found that a body of air surrounding the car moved with it, that is, the effective area of the car was more than that of the actual section.

No attempt was made to find the resistance due to impact of other than the front car. This impact amounted to a little, but as the openings between the cars were small, as shown by Fig. 5, it has been neglected on account of the difficulty attached to the measurement.

Graphical Logs.

Two graphical logs, Plate XIII, have been plotted, one for the east-bound and one for the west-bound resistance runs. The second and fourth runs are shown by dotted and the first and third by solid lines. The location, or distance, on the charts, was laid off horizontally on a scale of $\frac{1}{8}$ in. to 100 ft., and the light vertical lines give the 1 000-ft. marks. The numbers on these lines give the distances from the Church Street Terminal. The double vertical lines give the position of such locating points in the run as stations, entrances to tunnels, etc.

The lower lines are the speed curves, and give the speed of the train, in miles per hour, plotted as the average speed for the 15-sec. intervals, at the middle point of the distance covered in the 15 sec. These points are joined, and the resulting line represents the speed.

The next group gives the average net pressure, in inches, water gauge. The points are plotted directly above those for the speed, and the line joining these points gives the pressure curve. The magnitude of this pressure, when below $1\frac{1}{2}$ in., water gauge, is the average of the U-tube and corrected aneroid readings; when above $1\frac{1}{2}$ in., water gauge, it is the aneroid only. All the U-gauge and aneroid readings for the interval are averaged before the final average is taken.

The two upper curves are calculated from the pressure curves, and show the total air resistance that the train had to overcome. The points on these curves are calculated by multiplying the area of the car by the unit pressure, the result being total pounds.

East-bound Runs.—In the east-bound runs, as stated previously, the general direction of the train movement in the open was almost against the direction of the wind, and the air resistance, therefore, was increased over the west-bound runs, where the train was running with the wind. This resistance is shown very clearly on the air resistance curve in the open, Fig. 6, which will be described later.

It will be noticed that in the open the pressure curve, and conse-

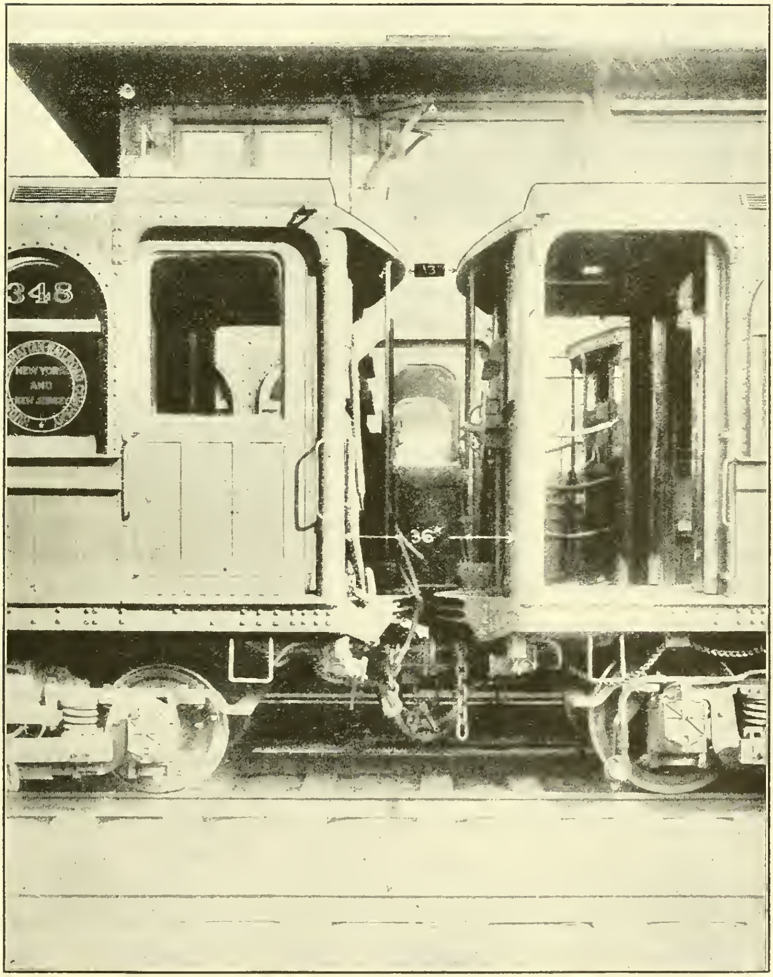


FIG. 5.—SPACE BETWEEN CARS.

quently the resistance, follows, with a few exceptions, the speed curve. The deviations in the pressure curve are probably due to errors in readings. On entering the rock cut, the speed had to be reduced on account of switches, and, therefore, no conclusion may be drawn as to the effect of entering the cut.

The two most salient features that are noticeable on the east-bound runs (Plate XIII) are the enormous jumps in pressure as the train enters the tunnels at the Portal and at the Pennsylvania Station for the trip under the river. When the large volume of air that has to be set in motion is considered, these pressures appear to be almost too low. The column of air in front of a train leaving the Pennsylvania Station for Church Street is about 6 000 ft. long, and if a section is taken the same as the car, 90 sq. ft., it would contain 540 000 cu. ft., and, at 62° Fahr., would weigh about 41 000 lb. This whole volume, if given an acceleration of 15 ft. per sec. (10 miles per hour per sec.), would require a total pressure of 19 200 lb. Of course, this is assuming that the air is solid and does not compress, and that it acts as a fluid. On account of the characteristics of air, it is impossible to calculate what this inertia pressure would be.

During the day, when traffic is heavy, this impact pressure will not be as high as that found in these trials. This is due to the fact that the train is not required to start this large volume of air from rest. From previous anemometer tests it has been found that during the day, when the trains are on 3 min. headway, the air in the river tunnels is rarely stationary, the velocity depending on the position of the train in the tube.

When a train enters the tube, an observer at the other end will notice an increase in the velocity of the air, which will increase as the train approaches, and reach its maximum as the train reaches his position. After the train passes, there is a very great rush of air (the filling of the vacuum caused by the train) for a short period, and this dies down as the train recedes; but, even after the train leaves the tunnel, some flow is noticeable, due to the momentum of the body of air, the effect of a train approaching in the same tunnel, but on the other side of an enlargement, and to the exhaust of trains in another tube which short-circuits at the station, in the case of an island platform, and enters the other tunnel.

Observations taken during the night, when the trains are running infrequently, such as every half-hour or less, show that the air will come to rest, and in some tunnels where the air passages at enlargements or stations permit it, will reverse, due to the movement of trains in the other tunnels. Therefore, the force necessary to overcome the air inertia will depend on the density of traffic.

At the time these tests were made, the traffic was very light, and it is fairly safe to assume that in the major part of the runs the air was at rest when the train entered the tunnel. Comparisons made with the entry pressures found in these runs and those for previous tests made in the day, show that the entering resistance when trains are frequent is about 85% of that found at night.

As the train enters the tunnel it is resisted by two air forces: one the impact and the other the static resistance due to moving the column of air. The first is independent of the length of tunnel or air columns in front of the train, while the latter varies directly with the length of the column. It is seen, therefore, that in long tunnels the total resistance does not depend on the speed alone, and, therefore, to some extent, the pressure line does not follow the speed line. This subject will be treated more fully later.

West-bound Runs.—On leaving Church Street Terminal (Plate XIII) and entering the tunnel, the same characteristic jump in pressure is encountered. Near the center of the tunnel a maintenance gang was at work, and the speed had to be reduced materially. This caused the drop in pressure shown for both runs. After passing this point the pressure increased with the speed, until the Pennsylvania Station was reached. Another gang was at work in Tunnel *G*, midway between Grove Street Station and the Portal. This caused another slow-down and a drop in pressure. The large increase in pressure just east of the Portal in test No. 2-*W* is due, to some extent, to the speed, but appears to be somewhat high.

In the open, the runs in this direction were almost before the wind, and this was very noticeable in Test No. 2-*W*, where at times, in place of a minus pressure at the rear, a zero pressure, and, in some instances, a plus pressure, was found.

Air Resistance in the Open.—On Fig. 6 has been plotted the air resistance encountered in the open at different speeds, in pounds. The lower curve (dotted) is the resistance when running with the wind, and the upper curve (light) is the resistance when running

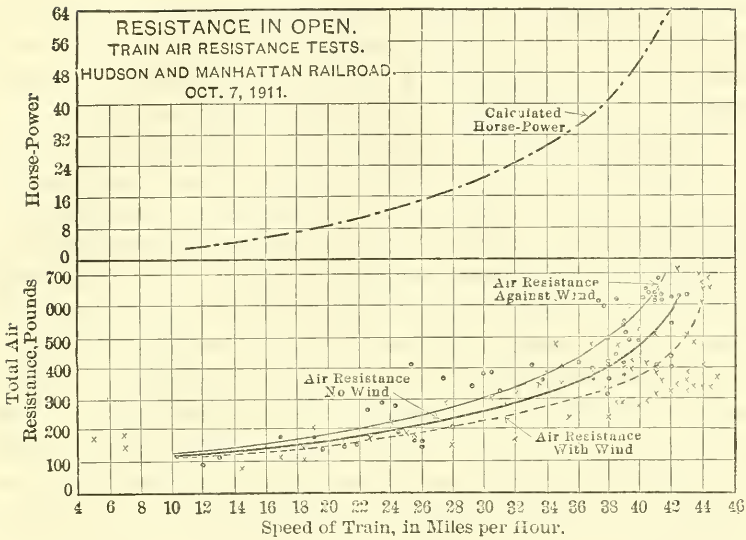

RESISTANCE IN OPEN.
TRAIN AIR RESISTANCE TESTS.
HUDSON AND MANHATTAN RAILROAD.
OCT. 7, 1911.
Calculated Horse-Power.
Horse-Power
Air Resistance Against Wind
Air Resistance No Wind
Air Resistance With Wind
Total Air Resistance, Pounds
Speed of Train, in Miles per Hour.

Fig. 6.

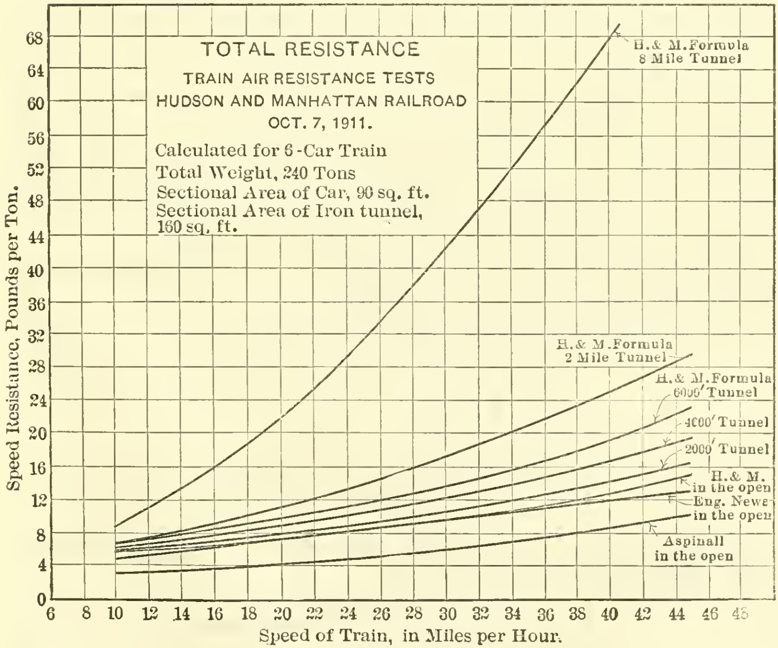

TOTAL RESISTANCE
TRAIN AIR RESISTANCE TESTS
HUDSON AND MANHATTAN RAILROAD
OCT. 7, 1911.
Calculated for 6-Car Train
Total Weight, 240 Tons
Sectional Area of Car, 90 sq. ft.
Sectional Area of Iron tunnel, 160 sq. ft.
H. & M. Formula 8 Mile Tunnel
H. & M. Formula 2 Mile Tunnel
H. & M. Formula 6000′ Tunnel
4000′ Tunnel
2000′ Tunnel
H. & M. in the open
Eng. News in the open
Aspinall in the open
Speed Resistance, Pounds per Ton.
Speed of Train, in Miles per Hour.

Fig. 7.

against the wind. The center curve has been drawn midway between these two, and gives the resistance with no wind. The curve on the upper part of the sheet is the corresponding horse-power, calculated from the center curve, or the resistance with no wind.

As mentioned before, the number of cars in the train would have some effect on the resistance due to impact on each coach end, but no account has been taken of this, except that any extra resistance due to this action would be included in the miscellaneous resistance. It is fairly safe to say that for cars of this cross-section, independent of the weight of the train and the number of cars, the resistance offered to the movement of the train by the air, would be as shown.

The curve is fairly flat until a speed of 30 miles per hour is reached, when the upward tendency is more marked. After 40 miles per hour the curve goes up nearly straight. This feature is borne out by other experiments and by the consideration that the impact, or speed air resistance, varies with some function of the square of the velocity.

Formulas.—The air resistance in the open must be wholly due to impact. Now, if it is assumed that the air which is acted on is given a velocity equal to that of the car, the results will be the same as if the car remained at rest and the air was blowing by the car at the train velocity. Then the magnitude of the pressure (*i. e.,* due to jet) bears to the weight of air flowing in a second the same ratio which the velocity per second of change in motion of the stream of air bears to the velocity generated by gravity in a second.

Working from this assumption, a formula has been worked out, the derivation being given in the Appendix.

The total resistance, in pounds, is as follows:

For speeds up to 32 miles per hour:

$$P = 0.0672\ D\ A\ V^2\ (1.772 - 0.0382\ V) \qquad (7)$$

For speeds greater than 32 miles per hour:

$$P = 0.0672\ D\ A\ V^2\ (0.0182\ V - 0.03) \qquad (8)$$

For cars of 90 sq. ft. sectional area, and for an air temperature of 52° Fahr., these formulas become:

For speeds up to 32 miles per hour:

$$P = 0.468\ V^2\ (1.772 - 0.0382\ V) \qquad (9)$$

For speeds greater than 32 miles per hour:

$$P = 0.468\ V^2\ (0.0182\ V - 0.03) \ldots\ldots\ldots\ldots\ldots\ldots (10)$$

in which P = Total air resistance, in pounds;
V = Train speed, in miles per hour;
D = Weight of the air, in pounds per cubic foot;
and A = Cross-sectional area of the car, in square feet.

Miscellaneous Resistances.—By miscellaneous resistances is meant the resistances other than air that appear when the train is running at constant speed on a level tangent. These are the resistances caused by journal friction, center-bearing friction, flange friction, etc., and depend to a great extent on weather conditions and temperature. As previously stated, any impact resistance due to gaps between cars in a train is included in this item.

On Fig. 7 a curve is shown marked "H. & M. in the open." This curve is the total resistance, in pounds per ton, to motion of the train when running at uniform speeds on a level tangent.

As mentioned before, it was very difficult to use the horse-power readings, because the train was seldom at constant speed, and, in the tunnels, so many other resistance-producing factors entered, such as grade, curve, etc., that a speed-resistance calculation was almost impossible; however, enough readings, at constant speed in the open, were available to plot, as shown on Fig. 7, the horse-power being reduced to its corresponding resistance in pounds per ton, and an allowance of 25% being made for motor and gear losses.

The air resistance in the open was then reduced to pounds per ton for the different speeds and taken from the total resistance; these values, therefore, are the miscellaneous resistance. This miscellaneous resistance, when plotted against the speed, is a straight line, the equation of which is:

$$P = 4 + 0.1\ V \ldots\ldots\ldots\ldots\ldots\ldots (11)$$

in which P = Miscellaneous resistance, in pounds per ton;
and V = Train speed, in miles per hour.

Total Resistance in the Open.—The total resistance to motion of trains in the open, at constant speed, is then the sum of the miscellaneous and air resistances.

The formulas, expressed in general terms, are:

For speeds up to 32 miles per hour:

$$R = 4 + 0.1\ V + \frac{0.0672\ D\ A\ V^2\ (1.772 - 0.0382\ V)}{W\ N} \ldots\ldots (12)$$

For speeds greater than 32 miles per hour:

$$R = 4 + 0.1\ V + \frac{0.0672\ D\ A\ V^2\ (0.0182\ V - 0.03)}{W\ N} \ldots\ldots (13)$$

For air temperatures of 52° Fahr., and for cars of 90 sq. ft. section, the formulas become:

For speeds up to 32 miles per hour:

$$R = 4 + 0.1\ V + \frac{0.468\ V^2\ (1.772 - 0.0382\ V)}{W\ N} \ldots\ldots (14)$$

For speeds greater than 32 miles per hour:

$$R = 4 + 0.1\ V + \frac{0.468\ V^2\ (0.0182\ V - 0.03)}{W\ N} \ldots\ldots (15)$$

in which R = Total resistance, in pounds per ton;
V = Train speed, in miles per hour;
D = Density of air, in pounds;
A = Cross-sectional area of car, in square feet;
W = Weight of car, in tons;
and N = Number of cars in the train.

Velocity Tests.—The velocity tests were made in order to determine the slip, or the volume of air which is not dispelled by the piston action of the train. The results obtained in the iron tunnels are reasonable and have been used, but those found in the tunnels of concrete section have not been used, as the Pitot tube did not extend far enough from the coach body, or did not reach beyond the zone of influence of the car. It has been found that, on all sides of the car, there is a certain depth of air which moves along with the train. If the tube does not extend out past this air, the results are useless.

The slip, or volume of air which is not expelled by the train, is shown by the left-hand group on Fig. 8. It will be noticed that the volume increases very rapidly with the speed of the train.

In the right-hand group on Fig. 8 has been shown the volume of air which is pushed in front of the train. This quantity is ascertained by subtracting the total slip from the total volume which would be discharged if the train filled the tube completely. The volumetric efficiency has also been plotted. By the volumetric efficiency is meant the ratio of the volume of air which is displaced to that which would be displaced if the train filled the tube completely.

It will be noticed that the volume of air displaced rises uniformly with the speed until near the end, when the curve shows a tendency to flatten. When one considers the great pressure difference between the front and rear of the train, this seems reasonable, for, as the speed increases, the plus at the front goes up and the minus at the rear goes down.

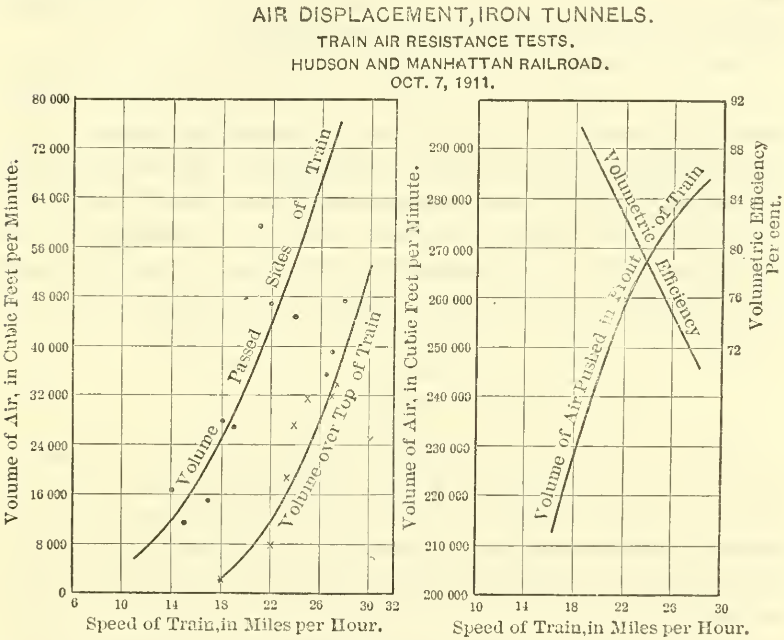


FIG. 8.

From the anemometer tests of 1909 and 1910 it was found that, for the average speed through the iron tunnels (3 min. for 6 500 ft., or 24 miles per hour), a steady flow of 190 000 cu. ft. per min. was developed. Now this 190 000 cu. ft. was the average taken for 2-min. readings, and of this 2 min., only 1½ min. were when a train was in the tunnel, the remainder being due to the flow induced by the passage of the train. Making this correction, it is found that, for a train moving at 24 miles per hour, the resultant velocity would be 1 590 ft. per min., and the corresponding volume would be 254 000 cu. ft. per min., which checks fairly well with the volume given on the curve for 24 miles per hour (13 000 cu. ft. per min. less).

Air Resistance, Iron Tunnels.

The air resistance in tunnels is made up of two components: that due to impact (*i. e.,* the jet action of the train striking the air) and the static pressure, which is due to the resistance offered to the movement of the displaced air by the walls of the tunnel. There is also the skin friction of the train passing through the air. This would not amount to very much, and has been neglected.

The total pressure on the train would be a function of the sectional area of the car and tunnel, and would not depend on the longitudinal area of the train, for any pressures that existed on one side, tending to force the wheel flanges against the rail, would be counterbalanced by the same pressure on the other side of the train.

Impact.—The impact, as has been shown by the discussion of the resistances in the open, is independent of the length of the tunnel, and is governed by the speed and the quantity of air deflected by the car. The derivation of this formula is given in the Appendix.

The total pressure, in pounds, is

$$0.0761\ V^2\ (12.75 - 0.1809\ V) \ldots\ldots\ldots\ldots (18)$$

This formula has been calculated between the limits of 18 and 28 miles per hour, but, without doubt, is applicable to speeds somewhat below and above these figures, say, from 15 to 40 miles per hour. Outside of these speeds, the variations are so considerable that it would not appear wise to use the formula. The curve for impact is shown on Fig. 9.

Static Resistance.—As the static resistance is the pressure necessary to overcome the friction of the air passing through the tunnels, it will depend on the length of the tunnel, the smoothness of the surfaces, the speed, the radius of the bends, the size of the outlets, etc. Therefore, the formulas derived will be applicable, strictly speaking, only to the tunnels in which the tests were made, and should perhaps be modified for other tunnels.

The formula for this component of the air resistance has been worked out in the Appendix, and is:

Total pressure in pounds =

$$0.000205\ L\ V^2 \ldots\ldots\ldots\ldots (21)$$

in which L = Length of tunnel between train and air outlet, in feet;
and V = Train speed, in miles per hour.

The length of the iron tunnels where these tests were made is such that three curves may be plotted; one for 2 000, one for 4 000, and one for 6 000 ft., and the three curves thus marked on Fig. 9 give the total air resistance for tunnels having approximately these distances between air exits; these curves being the total air resistance, or impact plus the static resistance.

Total Air Resistance, Iron Tunnels.—The total air resistance will be the sum of the impact and static resistances, or the sum of Equations (18) and (21), or

$$P_a = 0.0761\ V^2\ (12.75 - 0.1809\ V) + 0.000205\ L\ V^2 \quad (22)$$

Total Resistance, Iron Tunnels.—The total speed resistance for the iron tunnels, for a level tangent and a constant speed, will be the sum of Equations (11) and (22). The total resistance, in pounds per ton, will be:

$$P = 4 + 0.1\ V + \frac{0.0761\ V^2\ (12.75 - 0.1809\ V) + 0.000205\ L\ V^2}{W\ N} \quad (23)$$

in which P = Total resistance, in pounds per ton;
L = Length between air exits, in feet;
W = Weight per car, in tons;
and N = Number of cars in the train.

Air Resistance, Concrete Tunnels.

The total air resistance found in the concrete tunnels is plotted on Fig. 10, and also the calculated horse-power to overcome this resistance. These tunnels were shorter between air exits than the iron ones, the average distance being 3 000 ft.

Impact.—The formula for the impact resistance is derived in the Appendix, and is, in pounds:

$$P = 0.0745\ V^2\ (12.75 - 0.1809\ V) \quad (24)$$

This formula, when plotted for the different values of V, gives the curve shown on Fig. 10.

Static.—The vertical distance between the total air resistance curve as given and the impact curve, is the static pressure, and the formula, for the total resistance, in pounds, as worked out in the Appendix, is:

$$P = \frac{11.2\ V^2\ L}{700\ 000} \quad (26)$$

Total Air Resistance.—The total air resistance, in pounds, is the sum of the component resistances, or the sum of Equations (24) and (26), or

$$P = 0.0745\ V^2\ (12.75 - 0.1809\ V) + \frac{11.2\ V^2\ L}{700\ 000} \quad \ldots\ldots(27)$$

Total Speed Resistance.—The total speed resistance, in pounds per ton, will be the sum of Equations (11) and (27), or

$$P = 4 + 0.1\ V + \frac{0.0745\ V^2\ (12.75 - 0.1809\ V)}{W\ N} + \frac{11.2\ V^2\ L}{700\ 000\ W\ N} \quad \ldots(28)$$

Total Resistances.

On Fig. 7 the total resistances to train movement on a level tangent track, at constant speeds, have been plotted, assuming a 6-car train of 240 tons total weight. In all these curves the train speed has been plotted horizontally and the resistances, in pounds per ton of weight of train, vertically.

The lowest curve is plotted from the formula of Mr. J. A. F. Aspinall from his paper,* "Train Resistance," as follows:

$$P = 2.25 + \frac{V^{\frac{5}{3}}}{56 + 0.03\ l}$$

in which V = Train speed, in miles per hour;
P = Resistance, in pounds per ton;
and l = Length over coach bodies, in feet.

These tests were very carefully made in the open, but not with steel motor cars.

The next curve $\left(P = 2 + \frac{V}{4}\right)$ is one which was prepared by *Engineering News*.† This formula plots a straight line, and is found to coincide with the H. & M. curve from 18 to 33 miles per hour.

The third curve is the H. & M. in the open, and runs almost parallel with that of Aspinall, but is about 3 lb. above it. That may be due to the difference in the type of rolling stock.

The next three curves are plotted for H. & M. iron tunnels, 2 000, 4 000, and 6 000 ft. between openings.

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* *Minutes of Proceedings*, Inst. C. E., Vol. CXLVII, p. 155, in which it is given as $P = 2.50 + \frac{V^{\frac{5}{3}}}{50.8 + 0.0278\ L}$ for tons of 2 240 lb., and constants are changed for tons of 2 000 lb.

† October 31st, 1901.

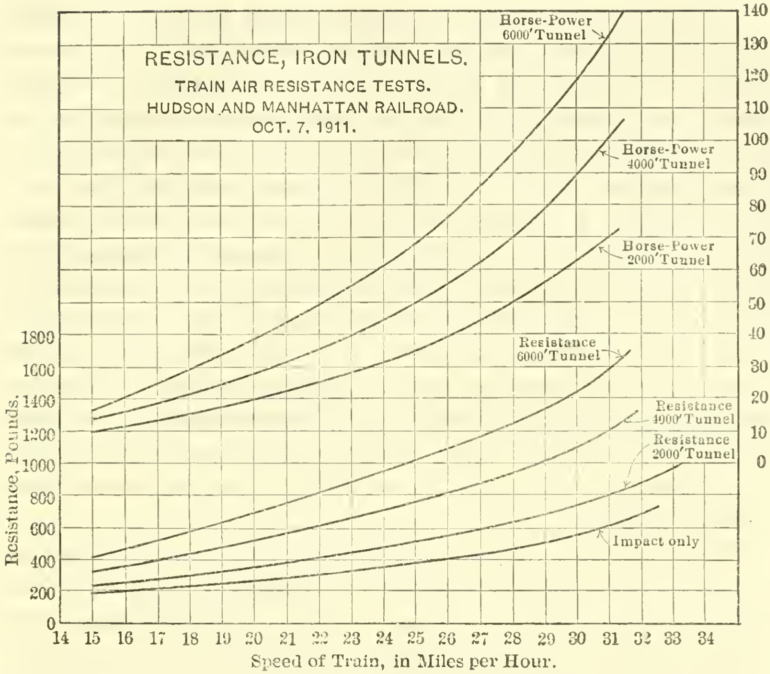
RESISTANCE, IRON TUNNELS.
TRAIN AIR RESISTANCE TESTS.
HUDSON AND MANHATTAN RAILROAD.
OCT. 7, 1911.
Horse-Power 6000′ Tunnel
Horse-Power 4000′ Tunnel
Horse-Power 2000′ Tunnel
Resistance 6000′ Tunnel
Resistance 4000′ Tunnel
Resistance 2000′ Tunnel
Impact only
Resistance, Pounds.
Corresponding, Horse-Power.
Speed of Train, in Miles per Hour.

Fig. 9.

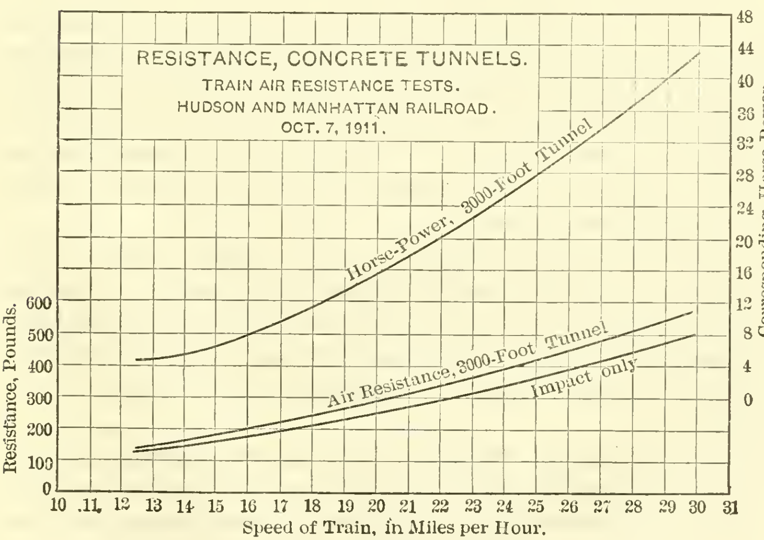
RESISTANCE, CONCRETE TUNNELS.
TRAIN AIR RESISTANCE TESTS.
HUDSON AND MANHATTAN RAILROAD.
OCT. 7, 1911.
Horse-Power, 3000-Foot Tunnel
Air Resistance, 3000-Foot Tunnel
Impact only
Resistance, Pounds.
Corresponding, Horse-Power.
Speed of Train, in Miles per Hour.

Fig. 10.

The next curve is plotted for an iron tunnel of 160 sq. ft. section and 2 miles between air exits. It will be noticed that, at 45 miles per hour, the total resistance is 30 lb. per ton. Assuming a 6-car train having a total weight of 240 tons, the total resistance will be 7 200 lb., and the corresponding electric horse-power, assuming a loss of 25% in transmission, would be 1 200, or would necessitate 200 e.h.p. per car. If it is assumed that each car has two 160-h.p. motors, there would be 120 h.p. available for other purposes than speed resistance, if the motors were worked up to their rating. If 20 lb. per ton is taken as the resistance per 1% of grade, the train would be capable of maintaining this speed on a 0.9% grade only. As this grade is far from being excessive in tunnel practice, it is evident that, in a tunnel of this description, the limiting speed is at 45 miles per hour for 6-car trains, and would very likely be lower. Another great difficulty would be met in acceleration, and the impact shock would be enormous.

The last curve on Fig. 7 is calculated for a tunnel 8 miles between air exits, such as a tunnel from Manhattan to Staten Island. It is not known that the formula would be applicable to a tunnel of that length, but it is certain that the resistance would be very high.

Assuming the same train make-up as before (6 cars, total weight 240 tons), and the total power as 1 920 e.h.p., then, at 40 miles per hour, the tractive power is 13 400 lb., or 56 lb. per ton; the calculated resistance is 68 lb., or the train could not reach this speed. At 35 miles per hour the tractive power would be 15 400 lb., or 64 lb. per ton. The limiting speed for a train of this kind in the tunnel, therefore, would be between 35 and 40 miles per hour on the level, working the motors to their capacity.

In a long tunnel of this kind, a factor which would enter into consideration would be the effect of such a high pressure on the ears of the passengers. In tunnels as long as those of the Pennsylvania Railroad at New York this is uncomfortable, but in a tunnel more than seven times as long, this discomfort would be very much more marked.

In connection with the air resistance of long tunnels, it may be of interest to note the results reported from the Simplon Tunnel.*

The electric locomotives used in this tunnel were from the Valtellina

* *Engineering*, Nov. 23d, 1906.

Railroad, and their section is reported as being two-thirds of that of the tunnel, or rather more than the section of the Hudson and Manhattan, which is 56% for iron and 54% for concrete. The weight of the locomotive is 62 tons, and the horse-power is from 900 to 2 300. It appears that about 400 h.p. more is required to haul a train at 70 km. per hour (43.5 miles per hour) through the tunnel than would suffice for it in the open. It was further found that on a gradient of 1 per 1 000, from the summit to Iselle Portal, the locomotive would not run down by gravity alone, but would with a train behind it. The fans installed at the Portal are reported to affect the resistance somewhat.

There are five remedies for cutting down this great resistance in long tunnels, where there is no way of making intermediate air exits:

(*a*) Blowing engines;

(*b*) More than one train in the tunnel at once;

(*c*) Wind shields on the trains;

(*d*) Smooth lined tunnels;

(*e*) Increase in area.

(*a*) *Blowing Engines.*—There is no doubt that the installation of blowing engines, or pressure fans, at one end and exhausters at the other, would reduce the air resistance materially, but the cost of the outfit and power would be very high, if the resistance was to be much reduced. On the other hand, the size and cost of the car motors would be correspondingly reduced, and, in some instances, this saving would balance the cost of installation.

(*b*) *More Than One Train in the Tunnel at Once.*—If more than one train is in the tunnel at the same time, the resultant total pressure will not increase in proportion to the number of trains. The impact on entry will be the same on each train, but the total static resistance will be about the same as if only one train was moving the air. In this way the total resistance on each train would be reduced.

(*c*) *Wind Shields.*—Wind shields of different kinds have been used in the open for cutting down the speed resistance of high-speed trains with varying success. In a tunnel the shield would reduce both components of air resistance, the impact as well as the static.

In Irminger's experiments* in towing models, he found that if the

* *Minutes of Proceedings*, Inst. C. E., Vol. CXLVII, p. 276.

resistance to motion of a plane at right angles to the direction of motion was represented by 100, a cone with its apex forward would have 42% resistance. Working from this, assume that a cone is built out at the front of the forward car, so that the angle at the apex is 90°; then, for a Hudson and Manhattan car, the diameter at the base would be 8 ft., and the distance from the apex to the front of the car 5 ft. 8 in. If it is assumed that the volume is the same as before, the volume of air which would be deflected would be 0.707 of the former volume. Then, as will be apparent from the derivation of the formula, the resistance would be cut down by this amount, or the resistance with the shield would be only 70% of that without, which, when compared with Irminger's tests, would seem reasonable. In this cone, however, there would be windows or other recesses or projections, therefore it may be said that only 25%, instead of 30%, would be the saving.

Assuming the same train as before, that is, 6 cars, 40 tons each, and a speed of 45 miles per hour, the resistance offered by a flat car surface, in the open, would be 15 lb. per ton, or an expenditure of 576 e.h.p. A saving of 25% by wind shield would be 144 e.h.p. This saving in power would be quite an item, and the first cost of motor equipment could also be reduced materially.

The saving by wind shield in tunnels would affect both components: the impact, by not deflecting so much air, and the static, by not forcing through the tunnel columns of air of such large section. The net resistance could then be assumed to be 25% of the plane front resistance. Then, for the same train as above, at 40 miles per hour, in a 6 000-ft. tunnel, the electric energy expended by a train having a flat front would be 655, and with a shield, 490, or a saving of 165 e.h.p.

In a 2-mile tunnel, at the same speed, the horse-power would be 860 for a plane front and 643 for a cone, or a saving of 217 e.h.p. For the very long tunnel (8 miles), the power without the shield would be 2 370 h.p., or 395 h.p. per car, and with the shield 1 780 h.p., or 297 h.p. per car, or a total saving of 590 h.p. It would then appear that for very long tunnels the wind shield is highly important, and in some cases might be absolutely essential.

There are several disadvantages in the use of a shield, the principal one being the inconvenience of always having, at the head end of the train, a coach equipped in this way, though, if locomotives were used, this would be of less importance. The ventilation by the piston action of the train would also be affected, as the volume of air dispelled would not be as great.

(*d*) *Smooth Lined Tunnels.*—The pressure necessary to force air through a duct depends to a great extent on the relative smoothness of its surface; the rougher the surface the more resistance is offered to the flow of air, and hence more pressure is required. This same question—smoothness in tunnel walls—will enter into the train resistance in tunnels, affecting the static component principally.

In the case of iron-lined tunnels, such as those of the Hudson and Manhattan Railroad, Fig. 3, there would be, every 2 ft., a flange, 3 in. wide, projecting about 6 in. toward the center of the tunnel, and at the center of each ring the erecting lug projecting above the inside of the iron. The air in passing must impinge on these projections, thus causing eddies and increasing the resistance. The highest velocities are found in the center of the tunnels, between the bench-walls, therefore, the air which actually strikes the walls and is broken up by these projections, is not moving at the greatest speed, and the total resistance due to this cause will not be as high as if the mean velocity of movement was uniform for the whole tunnel section.

The runs on October 7th were made through tunnels of two types of construction—the iron-lined and the concrete. The concrete has a slightly larger sectional area, 6 sq. ft., and the conditions as to air exits are not the same, therefore no direct comparison may be made in regard to the increase in resistance caused by the rougher surfaces.

Tunnel "A," starting a short distance west of Morton Street and extending for 3 500 ft., is an iron tunnel, lined with concrete in such a manner that no flanges or other projections extend beyond the concrete, and its section is essentially the same as the iron, 90 sq. ft. In some of the previous tests, runs were made through this tunnel, and the resistances were compared with those from the standard iron tunnels, where the other conditions as to length and air exits are as nearly alike as possible. This comparison would tend to show that the saving in total air resistance, due to the lining, was 20 per cent. This figure is only approximate, because the runs, of necessity, could not be made in the same tunnel, therefore other factors than the condition of the surfaces enter. This 20% saving does not appear to be too high, so that it is reasonable to suppose that there would be at least a 20% saving in power consumption by lining the tunnel.

Assuming a 2-mile tunnel with iron lining and the same make-up as before, let the average speed for the trip be 30 miles per hour, then the average speed resistance would be 13 lb. per ton, and the average power per trip 330 e.h.p. Assume that for 10 hours, 12 trains per hour are moved, for 8 hours 4 trains per hour, and for 6 hours 2 trains per hour, or 164 train movements per day per tunnel. This would be the expenditure of 2 720 kw-hr. per day of 24 hours, and at $\frac{3}{4}$ cent per kw-hr., the cost per day would be $20.40 for speed resistance alone. Now, if the tunnel was lined with concrete, 20% of this could be saved, or $4.00 per day, and this would be the interest at 6% on $24 400. Then, in order to be practicable, the cost of the extra lining should not exceed this amount; but it would be impossible to line an existing tunnel of this length for such a sum, so that, unless the tunnel was lined with concrete for some other reason, there would not be saving enough to balance the extra cost.

A combination of (*c*) and (*d*), or wind shields in a concrete tunnel, would mean a saving of 25% for the former and 20% for the latter, or a net saving of 45 per cent. Then, if (*a*) was used, this would mean, perhaps, 5% more, or 50% in all. Therefore, a combination of concrete-lined tunnels, with fans or blowing engines of large capacity at the ends, and the use of trains with wind shields, would appear to give a great saving in power consumption and first cost of electric equipment; and, in very long tunnels, it would give more satisfaction to the traveling public, because of the elimination of the pressure on the ears of the passengers.

(*e*) *Increase in Tunnel Section.*—As mentioned before, the cross-sectional area of the tunnel is the controlling factor in the air resistance offered to the train. As the section is increased, the resistance falls off, and approaches that found in the open. On account of mechanical clearances, the tunnel section could be reduced very slightly from that given, so that no calculation will be attempted for small areas.

Assume that the diameter of the tunnel is increased 1 ft., or from 16 ft. 7 in. to 17 ft. 7 in. The ratio of clearance to possible area in the standard tunnel is 74%; then for the 17 ft. 7-in. tunnel, the clear area would be 187 sq. ft. It is safe to say that, as the impact resistance depends on the quantity of air which strikes the car and is deflected, and as this volume depends on the area of the tunnel, the impact resistance would vary inversely as the section. Then, as the area of the standard bore is 86% of the assumed one, the impact in the latter case would be but 86% of the former, or 14% saving.

An examination of the formula for static resistance shows that any change in the diameter of the tunnel will affect both terms in the denominator—the K and the d. The K will change with the area, as this term modifies the V, or takes into account the difference between the train speed and the air velocity. It is reasonable to suppose that the K will increase in the same ratio that the area increases, for the volume of air which is pushed ahead of the train, and hence the velocity, will depend on the diameter of the tube.

The d, or pneumatic diameter, will also affect the static resistance, for, as it increases, the pressure will diminish. If the d is disregarded, then the decrease in static resistance will be in the same proportion as the impact. Then, in the case assumed, the total air resistance will be 86% of that in a standard tunnel.

From (d) it is seen that it would cost $20.40 per day for air resistance alone; then, if the saving is 40%, $2.85 per 24-hour day would be saved, or the interest at 6% on $17 300; but, if the extra foot is added for reducing resistance only, the $17 300 will have to cover the extra cost. A ring of the standard tunnel will weigh 5 670 lb. per lin. ft.; then, if the segments were no heavier, the addition would mean 340 lb. more per foot of tunnel, or, for the whole length, 3 590 000 lb. If the cost of the extra metal, grout, cement, labor, etc., was only 3 cents per lb., the total additional cost would be $107 000, or more than six times the saving in resistance. Any further increase in diameter would increase the extra cost in a much higher ratio than it would decrease the resistance cost, therefore, it would not be practical to increase the diameter for this purpose alone.

If, in place of single-track tunnels, tubes or subways of two or more tracks were used, the resistance would be materially reduced, but the ventilation due to the piston action of the train, would be seriously affected.

A train entering such a tunnel would displace and push ahead of it a volume of air which would be considerably smaller than that found in these tests, the larger part of the air passing back at the sides of the train and not being moved forward any great distance. This condition would exist until a train, moving in the opposite direction, entered the other end of the tunnel, when a counter current would be set up which would strike the first current, and the result would be a group of eddies.

In tunnels where there are two and four tracks, as in the Interborough Subway, and the traffic is very dense, there must be a considerable expenditure of energy due to these eddies, and if the outlet stations were farther apart, this would be still more marked. This action not only causes increased resistance, though probably not as much per train as in the single-track tubes, but eliminates the piston ventilation of the train, the air being shoved from side to side and given rotary motion instead of being expelled. This eddying and buffeting of the air permits it to absorb a large amount of heat from the operation of the trains, causing an increase in the temperature for the whole tunnel.

To ventilate by the action of the trains such a two- or four-track subway, in which trains are operated in contrary directions, appears to be practically impossible, and the ventilation of such tunnels must depend on the installation of fans, operated independently, which will remove the air at short intervals. The number and capacity of such fans involves a very heavy item, both in the first cost of installation and in the cost of operation. The enormous volume of air which is displaced and moved forward by the action of a train in a tube tunnel, such as described in the foregoing experiments, insures thorough and proper ventilation of the tunnel, provided the whole or a large portion of the air moved forward by the trains is removed entirely from the tunnel at certain fixed points. This has been the principle on which the ventilation of the Hudson and Manhattan tubes has been laid out, as has been described in the technical journals from time to time.

In these experiments it may be noted that the actual cross-section of the end area of the car is assumed as the area affecting the impact pressure. It is probable that this is not strictly the case, as there is undoubtedly a zone immediately exterior to the car which by frictional resistance draws the air along with it for a certain depth outside of the net structure of the car. It is practically impossible to determine the depth of this moving but gradually reducing zone of air; it would vary with its location along the body of the car, and in differing degree along the entire length of the train; consequently, for the practical purpose of the application of the results of these experiments, it is simpler to assume that the impact is transmitted on the net cross-sectional area of the actual car construction.

All the experiments and the working up of the results of these experiments have been carried out under the direction of George D. Snyder, M. Am. Soc. C. E., Principal Assistant Engineer, and by Mr. B. S. Murphy, who has been Assistant Engineer to the writer on the mechanical work throughout the construction of the plant and tunnels of the Hudson and Manhattan Railroad Company.

APPENDIX.

Train Resistance Tests, October 7th, 1911.

Air Resistance in Open.

From the assumptions made on page 998, we can write:

$$P : D\,Q :: (V_s - U_s) : g$$

or

$$P = D\,Q \left\{ \frac{V_s - U_s}{g} \right\} \quad \text{.....................(1)}$$

in which P = Total pressure on the car, in pounds;
V_s = Original velocity of jet, in feet per second;
U_s = Velocity of train, in feet per second;
g = Acceleration due to gravity = 32.16;
Q = Quantity flowing, per second, in cubic feet;
D = Density of air, in pounds per cubic foot;
and $D\,Q$ = Weight of air flowing in a second.

As the car is assumed to be at rest, $U_s = 0$. Then Equation (1) becomes

$$P = D\,Q\,\frac{V_s}{g} \quad \text{.........................(2)}$$

Let V = miles per hour, then

$$V_s = \frac{5\,280}{3\,600}\,V \quad \text{.........................(3)}$$

and $Q = A\,V_s$

where A = sectional area of the car, in square feet.

This would be true if the quantity deflected by the train varied directly as the sectional area, but this is not so, so this term will become

$$Q = K\,A\,V_s.$$

Where K is a variable, depending on the speed, substituting the value of V_s, in terms of V, gives

$$Q = K\,A\,\frac{5\,280}{3\,600}\,V \quad \text{.....................(4)}$$

Substituting these values of Equations (3) and (4) in Equation (2) gives

$$P = 0.0672\,D\,K\,A\,V^2 \quad \text{..............................(5)}$$

for an air temperature of 52° Fahr.

$D = 0.0776$, and, as $A = 90$ sq. ft., Equation (5) gives:

$$P = 0.468\,K\,V^2 \quad \text{..................................(6)}$$

This formula was then solved for values of K by substituting the values of P from the curve, and these were plotted against the corresponding speeds. When these values are plotted, the resultant figures approach the shape of the hypothenuses of two right-angled triangles, the apex of each being at 32 miles per hour.

The formulas for the curves are:

Up to 32 miles per hour:

$$K = 1.772 - 0.0382\ V.$$

Greater than 32 miles per hour:

$$K = 0.0182\ V - 0.03.$$

Substituting these values in Equation (5) gives, for the general expression for pressure:

Up to 32 miles per hour:

$$P = 0.0672\ D\ A\ V^2\ (1.772 - 0.0382\ V) \qquad (7)$$

Greater than 32 miles per hour:

$$P = 0.0672\ D\ A\ V^2\ (0.0182\ V - 0.03) \qquad (8)$$

Substituting the values of K in Equation (6):

Up to 32 miles per hour:

$$P = 0.468\ V^2\ (1.772 - 0.0382\ V) \qquad (9)$$

Greater than 32 miles per hour:

$$P = 0.468\ V^2\ (0.0182\ V - 0.03) \qquad (10)$$

Air Resistance, Iron Tunnels.

Impact.—(Page 1002.) Following the same reasoning as for impact in the open, we have

$$P = D\ Q\ \frac{V_s}{g} \qquad (2)$$

Now, $Q = V\ A$; but, from the velocity tests, it is known that the volume of the air deflected is not the product of the cross-sectional area of the car by the train speed, or that the A is not the same as the cross-section of the car. The area corresponding to the displaced volume was calculated and plotted against the train speed, and the resultant curve was found to be a straight line between the available speed limits of from 18 to 28 miles per hour. The equation for this line is

$$A_a = \frac{28 - V}{0.3704} + 115 \qquad (16)$$

in which A_a = apparent sectional area of the deflected air current at a velocity equal to that of the train.

$$\text{Also, } V_s = \frac{5\,280\ V}{3\,600}$$

$$\text{and } Q = V_s \left\{ \frac{28 - V}{0.3704} + 115 \right\} \text{ from Equation (16)}$$

$$\text{or, } Q = \frac{5\,280}{3\,600}\ V \left\{ \frac{28 - V}{0.3704} + 115 \right\}$$

Substituting these values in Equation (2) and simplifying, gives

$$P = D\ V^2\ (12.75 - 0.1809\ V) \qquad (17)$$

And, if the temperature of the air is 65° Fahr.,

$D = 0.0761.$

Then, $P = 0.0761\ V^2\ (12.75 - 0.1809\ V) \qquad (18)$

Static.—(Page 1002.) The second component of the total air resistance, or the static, would be governed by the same formula as used for the flow of air in ducts, or

$$p = \frac{L\ V_s^2}{K\ d} \qquad (19)$$

in which p = Corresponding resistance, in ounces per square inch;

V_s = Velocity of air, in feet per second;

d = Pneumatic diameter, in inches = $\frac{\text{Area of Duct} \times 4}{\text{Perimeter}}$;

L = Length from front of train to air exit, in feet;

and K = Constant.

Area of tunnel = 160 sq. ft.;

Perimeter = 605 in.;

$d = 152$ in.;

P = Total static pressure on car.

Then, as the area of the car is 90 sq. ft.,

$p = 0.00123\ P.$

V = train speed, in miles per hour.

Then, $V_s = 1.47\ V.$

Substitute these values in Equation (19),

$$P = 11.55 \frac{L\ V^2}{K} \qquad (20)$$

The lengths of the iron tunnels in which the tests were made were such that three lengths, or values of L, may be taken, namely, 2 000, 4 000, and 6 000 ft. The total air resistance found for each of these lengths is plotted on Fig. 9. The distance of each of these curves above the impact line will be the static resistance. Values of P were then taken from the curves and substituted for P in Equation (20) for the respective values of L, and K was solved. These values of K were averaged, and the result was 56 300. Then, substituting K = 56 300 in Equation (20), gave

$$P = 0.000205\ L\ V^2 \qquad (21)$$

Resistance, Concrete Tunnels.

Impact.—(Page 1003.) As mentioned previously, the velocity tests in the concrete tunnels were a failure, as the part of the total air deflected is not known, but, as the cross-sectional area of the concrete tunnels is 166 sq. ft. and that of the iron tunnels 160 sq. ft., it will be assumed that the slip is higher, in the ratio of $\frac{160}{166}$, or that the volume which affects impact is only 96% of that in the iron. Therefore, modifying Equation (18),

$$P = 0.96 \times 0.0761\ V^2\ (12.75 - 0.1809\ V)$$

or

$$P = 0.0745\ V^2\ (12.75 - 0.1809\ V) \qquad (24)$$

Static.—(Page 1003.) Using the same equation as for iron tunnels, Equation (19), or

$$p = \frac{L\ V_s^2}{Kd},$$

the area of the tunnel = 166 sq. ft., the perimeter = 608 in., and $d = 157$ in.

Substitute the values from Equation (19):

$$P = 11.2\ \frac{V^2\ L}{K} \qquad (25)$$

The value of K was worked out by substituting different values of P from the curve for a 3 000-ft. tunnel, from which K was averaged. The result was:

$$K = 700\,000.$$

Substituting this value in Equation (25) gives

$$P = \frac{11.2\ V^2\ L}{700\,000} \qquad (26)$$

This large increase in the value of K over that used for iron tunnels is not all due to the smoother surface, but depends also on the air exits.

DISCUSSION.

Mr. Gibbs. GEORGE GIBBS, M. AM. SOC. C. E.—In this interesting paper Mr. Davies refers to certain experiments which the speaker conducted in the Pennsylvania Railroad Company's New York Tunnels; these tests, however, were not to determine train resistance, as Mr. Davies apparently infers. While it has been the speaker's hope that he would be able shortly to make tests for this purpose, it has been impossible, thus far, to arrange for them. His experience in conducting such tests at various times during the past twenty years and, it may be said, under conditions more favorable for the purpose than those obtaining in the river tunnels in question, leads him to hesitate to undertake them without somewhat elaborate preparation.

Mr. Davies refers to the difficulties of eliminating from the observations the effect of variable conditions due to acceleration or retardation, changing each moment, almost, during a run, especially on a line involving frequent changes in grade. These have been experienced by the speaker also in interpreting the results of any tests of the kind. Even when using a dynamometer car, containing the most accurate instruments obtainable, it is difficult to eliminate from the records the effects of stored energy, which appear incessantly, continually accumulating or being given out between the cars of a train due to local acceleration or retardation in portions thereof from changes in track or motive-power conditions. In the case of tube tunnels, where the grades are not uniform, in fact where the profile is one long vertical curve, and where the run is relatively short, there would seem to be special difficulty in eliminating the variables above referred to; and, moreover, another variable is doubtless produced by currents, or eddies, in the moving air ahead of or following the train. The speaker's experiments indicate that eddies occur, even from very slight local obstructions along the tunnel walls, such as signals, etc., and at openings of cable manholes, ventilating ducts, and shafts. These eddies, it would seem, must have considerable influence on the air friction ahead of and at the side of the train, inasmuch as they are quite observable on a barometer placed in the car.

The tests in the Pennsylvania Tunnels were primarily for a determination of the ventilating conditions, and secondarily for an analysis of certain very sudden fluctuations of pressure which at high speed produce an effect in the ears of passengers in the trains.

The speaker's paper on the "New York Tunnel Extension of the Pennsylvania Railroad"* describes the emergency ventilating system installed in the tunnels. This system consists of a series of pressure fans arranged with the Churchill form of nozzles to blow air into the tunnels in the direction of traffic and thus induce a sufficiently

* *Transactions*, Am. Soc. C. E., Vol. LXIX, p. 226.

rapid movement of the air volume for satisfactory ventilation in the case of stalled trains. It was believed that the normal movement of trains which are scheduled at a high speed would produce sufficient piston action to give satisfactory ventilation, and it was the purpose of the tests to determine the facts accurately. Mr. Gibbs.

The Pennsylvania Tunnels are lined throughout with concrete, thus giving quite a smooth interior surface. The cross-sectional area of the river tube tunnels is 224 sq. ft., as compared with 166 sq. ft. in the Hudson and Manhattan Railroad Tunnels; but as the cars are larger than those of the latter railroad, the ratio of the cross-section of the car to that of the tunnel is 0.505, as compared with 0.54 for the Hudson and Manhattan Railroad. These conditions, therefore, in the two tunnels are nearly the same, except in portions of the Hudson Tunnels which are not concrete lined, where the ratio of car to tunnel is slightly greater. An important difference in the latter case is introduced by the presence of cast-iron segment flanges, which, according to the speaker's observation, are likely to cause a considerable amount of friction in the moving column of air.

Table 2 gives the maximum and average velocity of the air column caused by the passage of suburban trains at various speeds through the Pennsylvania Tunnels and shows that the air column attains a speed of from three-fourths to two-thirds of that of the train, depending on the speed of the latter; and the average speed of the air column for the entire time the train is in the tunnel is about one-half of the maximum and about one-third of that of the train.

TABLE 2.

Speed of train, in miles per hour.	Maximum observed air velocity, in miles per hour.	Average observed air velocity, in miles per hour.
20	15	7
30	24	12
40	30.5	15.5
50	36	18.5
60	39.5	21.0
70	43.5	23.5

The tests indicate that a suburban train moving at normal speed will push ahead of it, during its run from the portal to the station, about 1 000 000 cu. ft. of air, out of a total of 2 800 000 cu. ft. in the tunnel. After the train has left the tunnel it is found that the current of air in the tunnel remains in motion for about 5 min., but at constantly decreasing velocity, from which it is estimated that a suburban train of average length will replace about one-half of the air content of the tunnel by fresh air drawn in from the portal and from the shafts. Closing the intermediate shafts was found to reduce the ventilation somewhat, but not greatly.

Mr. Gibbs.

Satisfactory ventilation in a car requires about 30 cu. ft. of fresh air per minute per passenger, calling for 50 cu. ft. of air to be renewed outside of the car; which means that during the rush-hour traffic a complete renewal of the air in the tunnel is required every 20 min. If, therefore, the movement of one train renews one-half the air in the tunnel, the passage of a train every 10 min. will produce satisfactory ventilation. Rush-hour service, however, means a train spacing of about 2½ min. apart; it follows, therefore, that during this worst period

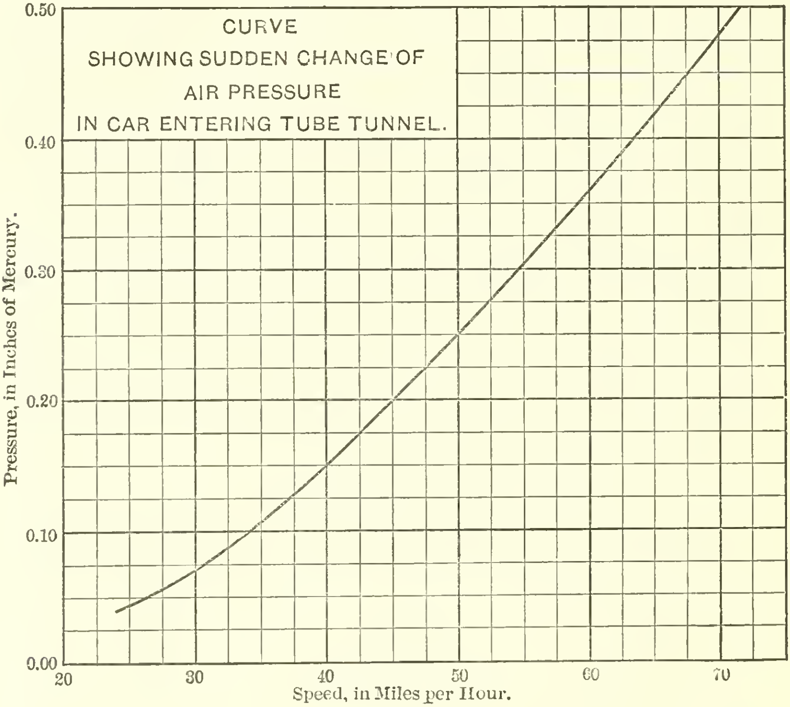

FIG. 11.

four times the required ventilation is produced by the train movements, without the help of the ventilating fans.

The fluctuation of air pressure inside the cars of a suburban train was observed by placing a sensitive barometer in the motorman's compartment at the head, and the air pressure in the tunnel itself was obtained by similar barometers placed on the bench-walls near the entrance and at the shafts. The curve, in the diagram, Fig. 11, shows the maximum sudden fluctuation or change in air pressure in the car as it passes the tunnel portal and at the open shafts, at various speeds.

These pressures build up quite suddenly, and fall again as rapidly, after passing the points mentioned, nearly to normal and only slightly above the atmospheric pressure outside of the tunnel. It may be thought that this latter result is inconsistent with the theory that there is a constant pressure ahead of the train due to the fact that the air column is not moving at the same speed as the train, and to the fact that pressure must be produced in overcoming friction. This indicates the great difficulty encountered in obtaining consistent results from such tests; in fact, the net pressure in the cars seems to be the resultant of that produced by the eddying currents around, and the suction at the sides of, the cars, as well as by inertia and friction. Therefore, all that can be decided definitely from the tests is that a violent surge is produced as the train enters the portal, and as it passes the shafts, and that the effect at the open shafts is nearly as great as that at the portal. Closing the shaft openings reduces the surge, but does not eliminate it entirely, in fact, it is found that even the small air duct and stairway openings produce local disturbances, and these cannot be entirely eliminated as long as there is any variation in the cross-section of the tunnel. Mr. Gibbs.

It is also a curious fact that fluctuation of pressure inside the cars is noticeable, although somewhat reduced in amount, when all the windows and ventilators are shut. This shows that a very small addition to the air volume in the car causes the effect—that it is a "barometric" and not a wind pressure which is felt in the ear. Probably, also, the slight deflection of the car sides from an outside variation in pressure contributes to the result. At slow speeds, say, up to 30 miles an hour, there is hardly any noticeable effect in the ears of persons in the train, and the disturbance becomes disagreeable only when speeds of 50 or 60 miles an hour are attained, and the effect is only momentary.

The speaker is indebted to the Test Department of the Pennsylvania Railroad for many of the test results used as the basis of this discussion.

George H. Pegram, M. Am. Soc. C. E. (by letter).—There has not been time, since this paper came to the writer's notice, to make the necessary tests for such a discussion as the subject merits. It is a valuable and timely addition to our knowledge of tunnels. The magnitude and difficulty of the task of deducing formulas for air resistance, which will serve as guides in the design of tunnels, become very apparent when tests are attempted. Mr. Pegram.

The writer has made several tests in the East River tubes of the Interborough Rapid Transit Company between the Battery shaft in Manhattan and the Willow Place shaft in Brooklyn. When tabulated, however, they contain irregularities which cannot be explained, and they are presented in the light of a study to be interpreted in connec-

Mr. Pegram.

tion with the manner of making the tests and should not be accepted as final results. These tests are shown in Table 3. Fig. 12 is a profile of the tunnel.

The tests indicate that Mr. Davies' Formula 22, page 1003, will probably give good results for total pressures in tunnels of this cross-section, but they show nothing as to the factor, *L,* which is an important part of this formula. *L* is the distance between the train

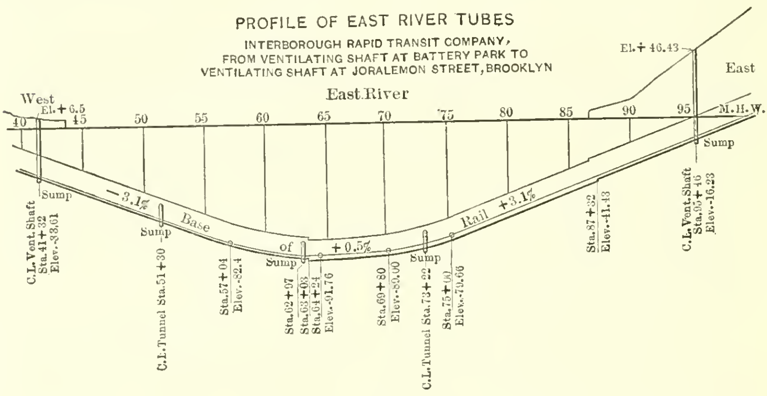

FIG. 12.

and the air outlet. It would seem that this factor should be connected with the total length between the air outlets, because the column of air following the train has an effect similar to that preceding the train.

The Battery Tunnel is 5 400 ft. long, between the ventilating shafts in New York and Brooklyn, and has a cross-section nearly identical with that of the Hudson and Manhattan tube shown by Fig. 2, except that a lining of concrete fills the spaces between the flanges of the iron, giving a smooth bore. The clear sectional area is 158 sq. ft., as compared with 160 sq. ft. in the Hudson tube. For 2 400 ft. of this length, however, a reinforced concrete lining is used, which contracts the area in this portion to 145 sq. ft. The section of the cars is 90 sq. ft., the same as those of the Hudson and Manhattan. The ventilating shafts at the ends are 14 ft. wide in the length of the tunnel, and 41 ft. long, with a diaphragm in the middle separating the tubes.

There are openings between the tubes at the three sumps, but these are kept closed with doors flush with the tunnel walls. The ventilating shafts provide free outlet of air to the surface. The conditions would seem to present a good opportunity for getting valuable results.

Mr. Pegram.

TABLE 3.—AIR RESISTANCE TEST. EAST RIVER TUBES OF INTERBOROUGH RAPID TRANSIT COMPANY.

Between Ventilating Shafts at Battery Park, Manhattan, and Willow Place, Brooklyn.

Battery Park Shaft, Station 41 + 44, Elevation — 33.68′

Willow Place Shaft, " 95 + 46, " — 16.23′

Clear Sectional Area of Tubes, Station 41 + 44 to 63 + 24 = 158 Sq. Ft.

" " " " " " 63 + 24 to 87 + 32 = 145 " "

" " " " " " 87 + 32 to 95 + 46 = 158 " "

Sectional Area of Car = 90 Sq. Ft.

North Tube to Manhattan.								South Tube to Brooklyn.						
Station.	Number of cars.	Speed of train, in miles per hour.	Pressure, in Pounds per Square Foot.			Total resistance to train.		Station.	Number of cars.	Speed of train, in miles per hour.	Pressure, in Pounds per Square Foot			Total resistance to train.
			Front of train.	Rear of train.	Total.						Front of train.	Rear of train.	Total.	
52 + 00	8	29.8	4.7	2.1	6.8	612	Observations taken on bench.	46 + 00	10	30.6	6.0	4.9	10.9	981
52 + 00	8	27.0	5.4	2.1	7.5	675		47 + 00	10	33.6	12.8	5.2	18.0	1 620
58 + 60	8	34.2	2.3	6.0	8.3	747		55 + 00	10	34.9	14.3	4.9	19.2	1 728
63 + 50	8	40.0	3.9	6.4	10.3	927		57 + 00	10	38.8	10.5	9.0	19.5	1 755
63 + 50	8	36.0	3.8	7.5	11.3	1 017		60 + 00	10	42.6	12.8	10.5	23.3	2 097
66 + 25	8	36.0	5.3	8.2	13.5	1 215		60 + 00	10	42.6	11.3	9.0	20.3	1 827
66 + 25	8	36.0	5.3	8.6	13.9	1 251		60 + 00	10	41.6	10.5	11.3	21.8	1 962
66 + 25	8	31.2	4.5	10.9	15.4	1 386		60 + 00	10	35.6	8.6	6.4	15.0	1 350
70 + 00	8	48.4	5.6	13.9	19.5	1 755		60 + 00	10	32.3	9.8	5.6	15.4	1 386
70 + 00	8	39.0	7.1	2.3	9.4	846		63 + 00	10	44.8	9.8	8.3	18.1	1 629
70 + 00	8	36.9	6.0	7.5	13.5	1 215		63 + 00	10	38.8	6.8	9.8	16.6	1 494
79 + 75	8	43.8	12.7	11.3	24.0	2 160		63 + 00	10	34.9	4.5	6.4	10.9	981
79 + 75	8	43.8	8.3	5.3	13.6	1 224		68 + 00	10	29.6	7.9	3.0	10.9	981
79 + 75	8	35.0	8.3	6.5	14.8	1 332		71 + 75	10	38.8	8.3	5.3	13.6	1 224
79 + 75	8	31.9	8.3	6.8	15.1	1 359		71 + 75	10	34.9	6.0	6.0	12.0	1 080
90 + 00	8	38.9	9.0	4.5	13.5	1 215		71 + 75	10	34.9	2.3	6.0	8.3	747
90 + 00	8	31.9	5.3	8.2	13.5	1 215		71 + 75	10	32.3	6.8	5.6	12.4	1 116
94 + 50	10	39.7	10.5	1.2	11.7	1 053		71 + 75	10	31.7	2.3	5.6	7.9	701
94 + 50	10	32.0	10.5	1.9	12.4	1 116		71 + 75	10	29.6	6.0	4.5	10.5	945
94 + 50	10	21.0	3.0	0.8	3.8	342		91 + 10	10	25.7	3.8	10.5	14.3	1 287
79 + 75	8	37.0	7.9	16.0	23.9	2 151	Observations taken on train.	60 + 00	8	42.0	9.4	14.6	24.0	2 160
								60 + 00	8	34.0	5.0	12.7	17.7	1 593

Mr. Pegram. The tests were made during the regular operation of the tunnel, between 1 P. M. and 4 P. M. on May 10th, 13th, and 14th, 1912. Three observers were located at a point on the duct bench at the side of the tube. One faced the approaching train, reading an aneroid barometer held steadily in the hand, another recorded the readings, and a third, with a stop-watch from which the speed was calculated, took the time required for the train to pass. Frequent readings were made, but those used in Table 3 were the passing of the head and rear of the train, respectively. Readings of the barometer were also taken at the shaft and at Bowling Green Station.

A few tests were made with the aneroid barometer on the front and rear of the train, with the instrument in the position used in the Hudson and Manhattan tests. The speed in these cases was taken by an observer with a stop-watch on the tunnel bench at the stations noted. These results are shown in Table 3.

The most discouraging feature of the observations was the fact that, during the approach of a train, sometimes at a distance of a quarter of a mile, the instrument would show a pressure twice as great as observed as the head of the train passed the barometer, and this pressure would rise to three times the head pressure at a distance of somewhat more than a train length; also, an observer standing in the ventilating shaft at one side of the track would observe a normal pressure, while, in the middle of the track facing an approaching train, he would note an added pressure of about 5 lb. per sq. ft., doubtless caused by the momentum of the air passing along the line of the tunnel.

It would seem as if the relative areas of the tube and the car should be introduced into the formula to make it generally applicable.

It is evident, further, that tests must be made in tunnels of different sizes and with the various conditions of outlets, in order to deduce a general formula, and it is incumbent upon engineers, generally, to supply such data as they can.

The proposed wind shield in front of the train will probably have very little effect in a tunnel for want of a surrounding space into which to deflect the air.

The Battery tubes are provided with blowers at the ends, and it is possible that their efficiency in reducing air resistance might in a measure be determined, although the blowers are installed simply for the purpose of blowing back smoke from either end in case of fire, to allow safer exit to passengers.

Mr. Churchill. CHARLES S. CHURCHILL, M. AM. SOC. C. E.—Although this subject is very interesting, and Mr. Davies has the thanks of the Profession for having undertaken these tests, it is extremely important that conclusions too far-reaching be not drawn at this time. The author states that this tunnel is especially small, relative to the cars.

Although it is not large, there are several steam railroad tunnels which are no larger. For example, Elkhorn Tunnel, on the Norfolk and Western Railway, has practically the same ratio; and in ventilating that tunnel the method of driving the air ahead of the train was adopted. One can sit in a car at the rear of a train and fresh air is felt coming into it through the operation of the fans. Mr. Churchill.

In other cases—at the Washington Tunnel, and also at Big Bend Tunnel, on the Chesapeake and Ohio Railway—the air is forced against the train. In the Washington Tunnel no smoke comes out at the station end, and yet the train drives a certain amount into the large tunnel area which is behind the station, as the train comes out into that area. All the smoke, however, is forced out finally at the other end of the tunnel.

These general points are mentioned in order to make clear the fact that the resistance decreases rapidly after the air is put in motion. The very principle of ventilation that has been used in these tunnels takes this into account; and it is a fact that when air is forced into a tunnel at one end, all of it goes out at the other, and there is no large cumulative resistance.

Now, it does not make any difference whether the tunnel is 1 000 or 5 000 ft. long, the resistance is not very great. Of course, it varies with the length, but it is questionable whether the pressure curve takes the extreme shape shown in the upper curve of Fig. 7.

The City and South London Railway Tunnel, under the Thames River and beyond it, has a section 1.4 times the train section. One section of the Hudson and Manhattan Tunnel is 1.6 times that of the train. Observations in the City and South London Tunnel in 1906 showed that very little ventilation was secured by the train, and about 17 000 cu. ft. only were moved for a period of 1 min. upon the passing of the train, only about 7 000 cu. ft. within ½ min. after the passage of the train, and only a small quantity moved through the shafts.*

These points are mentioned, not for the purpose of giving any exact data, but simply because they have some bearing on any conclusion that may be drawn from this investigation.

The tests by Mr. Davies have been started in directions which should lead to some very valuable results, but it is undesirable to draw conclusions too quickly, and it is important that further tests be made in order to develop this subject.

L. J. Le Conte, M. Am. Soc. C. E. (by letter).—This paper is an extremely valuable contribution to the records of the Society. All the useful facts are conveniently concentrated on the diagrams, and the Mr. Le Conte.

* *Transactions*, Am. Soc. C. E., Vol. LVII, p. 227.

Mr. Le Conte. results speak volumes of reliable information, the full value of which cannot now be over-estimated.

It has occurred to the writer that another comparison might be of some interest. As Fig. 9 gives resistance curves for 2 000, 4 000, and 6 000-ft. iron tunnels, an interpolation between the 2 000 and 4 000-ft. curves would give a curve for a 3 000-ft. tunnel, and the results might be compared with those for a 3 000-ft. concrete tunnel. This has been done, and the results are quite remarkable, as shown in Table 4.

TABLE 4.—Comparison of Resistances in 3 000-Ft. Iron and Concrete Tunnels.

Speed, in miles per hour.	3 000-ft. Tube Tunnel.	3 000-ft. Concrete Tunnel.	Difference.
15	255 lb.	180 lb.	75 lb.
20	438 "	280 "	158 "
25	643 "	420 "	223 "
30	920 "	575 "	345 "
32	1 200 "	750 "	450 "

The author, however, states that the air outlets are not the same in the two cases, and, therefore, a strict comparison cannot be fairly made. Nevertheless, the results are very interesting as showing, in a general way, the great effect of the interior roughness of tunnel linings.

The author speaks of the net end area of the car not being the effective area producing resistance. This same difficulty permeates the entire science of hydraulics and pneumatics. It arises, of course, from the natural viscosity of water and air, particularly at high velocities. The effective area of the resisting object increases with the velocity, hence the resistance increases faster than the simple square of the velocity, as is usually assumed. Of course, the wind shield certainly would play a very important part in reducing this effective area, especially if it were built on true parabolic lines. If it is properly built to suit the highest speed of the trains, the saving in resistance will be fully 30%—that is to say, when the free space is 44% of the tunnel section.

Mr. Davies. J. V. Davies, M. Am. Soc. C. E. (by letter).—Mr. Gibbs speaks of the influence of eddies on the air currents due to local obstructions along the sides of the tubes. The anemometer tests, mentioned in the paper, indicate that the velocity of the air is greatest in the center of the cross-section of the tube and least at the sides, where these obstructions are located, and, as they are not frequent, their influence on the ultimate air resistance cannot be great, although they do cause eddies and momentary fluctuations in pressure.

In regard to Mr. Pegram's reference to the factor, L, in Formula 22, it should be noted that, in the body of the paper, L is given as the length between air exits, in feet, while, in the appendix showing the derivation of the formula, it is given as the length from front of train to air exit, and there it is explained that the constant in the formula is obtained from the average results in tests of resistance in tunnels 2 000, 4 000, and 6 000 ft. long, respectively. The formula, therefore, is at the best an approximation, and represents the average air resistant in a tunnel L feet long, instead of the resistance at a point L feet from the head of a train to an outlet. These results, obtained with the formula, therefore, will be too high at one end and too low at the other. The air that immediately follows a train is at a very high velocity, but it decreases quite rapidly as the train departs, while the air ahead of the train has a very much higher velocity for a much longer period, and, therefore, the distance ahead of the train was used in the derivation of the formula as having a more controlling influence than that in the rear. Mr. Davies.

With respect to Mr. Pegram's suggestion that the relative areas of the tube and the train should be introduced in the formula: The relation of the area of the car to the area of the tunnel is taken care of in the empirical constant in the static resistance formula, which may vary with a change in the ratio between the area of car and area of tube.

Mr. Churchill states that the air resistance decreases rapidly after the air is put in motion. This statement, when applied to a tube tunnel, is correct to a very limited extent only, the very fundamental principle of fluid friction indicating the contrary, as the resistance to air passing through a duct will increase as the square of the velocity. The tests described in this paper indicate that it does make a difference whether a tunnel is 1 000 or 5 000 ft. long, particularly where high velocity is used. Of course, it is not claimed that these tests justify the acceptance of the results given in Fig. 4 when applied to a tunnel 8 miles long, this being given only to indicate the need of further experiments before a project for a tunnel of this length, for high speeds, should be undertaken.

In regard to Mr. Churchill's reference to the observations in the City and South London tunnel, which indicate that very little ventilation was secured by the train: It is likely that this is caused by the slow movement of the trains and the lack of proper air outlets. As the experiments in the Hudson and Manhattan tunnels indicate, the fans could be dispensed with entirely if it were possible to have sufficient inlets and outlets to the surface, and, in fact, portions of these tunnels are entirely ventilated by the piston action of the trains, the fans being installed only for emergency use in case of fire, or

Mr. Davies. "blow-out," causing a large volume of smoke with no trains in motion to expel it. The value of the piston ventilation is verified by Mr. Gibbs' tests in the Pennsylvania tunnels.

It appears to the writer that there are two matters which are of interest enough to refer to in connection with this subject, although, strictly speaking, they are only akin to the subject; they are the personal sensations felt by employees within the tunnels and by passengers riding in cars within the tunnels, due to the flow and resistance of air caused by the movement of trains. Immediately after the commencement of operation of the Hudson and Manhattan Railroad it was found by employees that there was little need to watch for the headlights of trains, as they could tell instinctively by the movements of the currents of air when trains were approaching, and by the velocity of the currents the nearness of the train. Taking one of the downtown tunnels as an illustration: it is immediately apparent to a person standing in the tunnel, near the Pennsylvania Station, Jersey City, the moment a train enters the tube under Fulton Street, New York, or, in the reverse direction, to an observer at the end of the tube at Cortlandt Street, New York, the starting of a train from the Pennsylvania Station, Jersey City. These points are more than one mile apart. To an observer, the movement of air is very gentle at first, but as the train comes nearer the velocity obviously becomes greater, up to the point of immediate approach of the train. This is so marked that at junction points employees instinctively know, from the movement of the air currents, from which direction a train is approaching.

The influence on passengers riding in coaches is a matter which caused very considerable comment by commuters on the Long Island Railroad immediately after the opening of the Pennsylvania tunnels. The initial schedule of operation called for very high speeds through these tunnels, and trains entering from the Long Island City approach, at high speed, on a descending grade into the tunnels, would strike the portal at high velocity with an initial pressure of air which was found to be very troublesome to the ear-drums of those riding in the trains. The writer has been informed that, in consequence of this, the initial operating speeds have been quite materially reduced when trains enter the portal. At the speeds usually operated in the Pennsylvania tubes, and also in the Hudson and Manhattan tubes and the Interborough's Battery tunnel to Brooklyn, it has been found from repeated observations that there is a pressure of $\frac{1}{10}$ in. of mercury, by barometric reading, under ordinary conditions of operation, and that this pressure produces a most distinct sensation on the ear-drums of passengers. There is a peculiar fluctuation of these air pressures at points of passing outlets, such as approaching a vent shaft or enlargement. The pressure will bank up with a moving train to $\frac{1}{10}$ in. above the normal, and, immediately before passing such an outlet, will drop to slightly below normal, indicating a partial vacuum at the rear of the train, and then again a sudden ascent of pressure after the outlet, or enlargement, is passed, which is equivalent to again entering the portal of the tunnel. This fluctuation causes sudden "blows" (so to speak) upon the ear-drums, which are probably not noticed by those in the habit of traveling daily in the trains; but, to strangers or those who are not accustomed to such daily traveling, it is distinctly unpleasant. This physiological effect indicates very markedly to the observer the principles of air resistance which are outlined in the paper. Mr. Davies

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ADDRESS AT THE 44TH ANNUAL CONVENTION, SEATTLE, WASHINGTON, JUNE 25TH, 1912.

By John A. Ockerson, President, Am. Soc. C. E.

Sixteen years ago this month the Annual Convention of the American Society of Civil Engineers was held for the first time on the Pacific Coast, and our second visit has perhaps been unduly delayed, although we have for years looked with longing eyes toward the time when a meeting could again be held on these hospitable shores.

There is a tradition among the good people of this Western World that those who make a third visit become so enamored of the advantages, the attractions of the climate, the material resources, and the people, that they refuse thereafter to leave it, but in turn become enthusiastic advocates of life in the Far West.

Perhaps it is just as well, therefore, that the intervals between meetings on the Pacific slope are somewhat long.

It is, however, well worth a long journey to any of our Annual Conventions and our Annual Meetings, to exchange greetings with old friends and acquaintances, and to find new ones. This applies with equal force to the whole country, be the distance from the Society House to the place of gathering great or small.

My membership in this Society dates back to the Twelfth Annual Convention, which was held in St. Louis in 1880.

Since that time thirty-one presidents of the Society have delivered their annual addresses, covering the whole range of duties and relations of members to the Society, the purposes of the Society, its influence in elevating the Profession in public esteem as the peer of the so-called learned professions, and its important function in establishing the unity of the Profession of Civil Engineering, although many organized engineering specialties have developed other societies with large and active membership.

The important place which the engineer has occupied and will continue to occupy in the development of our country, by directing the "great sources of Power in Nature for the use and convenience of Man" has been repeatedly discussed in convincing manner.

Codes of ethics and laws to govern the practice of Civil Engineering have been ably treated on several occasions.

Some of the presidential addresses were prophetic in their estimate of what the future would bring forth; and the progress in the field of engineering cannot be better illustrated than by citing a few of the predictions that have been more than realized, even during the comparatively brief period of my membership in this Society.

Some of you will doubtless remember that the late Octave Chanute, who was Vice-President of the Society in 1880 and later became President, was the American pioneer in efforts to solve the problem of aerial flight by man.

In the Presidential address,* prepared by him in that year, he stated:

"I suppose you will smile, when I say that the atmosphere yet remains to be conquered; but wildly improbable as my remarks may now seem, there may be engineers in this room, who will yet see men, safely sailing through the air."

He lived to see the fulfillment of this prophecy himself, so far as relates to the possibility of navigating the air; but there is evidently still much to be desired as to the safety of those who travel in aeroplanes, as well as "they that go down to the sea in ships."

Another President of the Society, a year later, quoted the prediction of Sir William Thomson, that the time would come when the Falls of Niagara would be largely utilized for light and power over a large area of North America; that a half-inch copper wire would transmit 21 000 horse-power to Montreal, Boston, New York, or Philadelphia.

The remarkable developments in the application of electricity for the propulsion of railway trains and street cars, for lighting cities,

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* *Transactions*, Am. Soc. C. E., Vol. IX, p. 255.

for electric furnaces, and for motive power of various kinds, all demonstrate that the prediction made has been greatly exceeded by actual progress in many lines.

Water-power plants for the development of electric energy of far greater capacity than were even dreamed of by Sir William Thomson are now in operation or in process of construction in many parts of our country.

One of the greatest of these is the power plant now under construction at Keokuk, where a concrete dam across the Mississippi River will result in developing 200 000 horse-power by converting the energy of the flowing stream into electric currents which may be transmitted to distant points for use.

In another address of this period, the development of elevators for successful use in high buildings was heralded as a remarkable example of progress in engineering, while they have long since come into such general use as to be no longer a luxury, but a necessity, required in every building of even moderate height.

In the vicinity of New York, ferry-boats have been largely replaced by great bridges and tunnels, which are to-day striking examples of the skill of the engineer. In the same locality enormous sums are being expended on terminals to facilitate the movement and interchange of traffic. Neither is the health of the people of our greatest city lost sight of, as is shown by the gigantic work now in progress which will provide an ample supply of water from sources free from pollution.

A similar work of large proportions is nearing completion for the City of Los Angeles, the details of which will be presented to the Society at this Convention.

Work on the Panama Canal, the greatest project of this century, is progressing rapidly. The problem of an Isthmian Canal, which received the serious consideration of several nations for many years, with a great diversity of opinion as to its proper location, its dimensions, its character—as to whether it should be at sea level or be constructed with locks—is soon to be realized as a successful work of American engineers, which will be of vast benefit to the world at large, even if not profitable to us as a business venture. Less than twenty years ago, the President,* in his annual address, stated that, in his opinion, the day was not far distant when the United States would

* William P. Craighill, President, Am. Soc. C. E., *Transactions*, Vol. XXXI, p. 568.

construct, own, and hold a ship canal across the Isthmus, and that then, perhaps, our domain would be extended to make that canal part of our southern boundary.

The great irrigation works which are converting the desert into fertile fields are essentially projects of to-day that engage the highest skill and energy of the engineer.

A remarkable feat in railway building is to be found in the road over which we have been carried in comfort and safety to this beautiful city.

Rivers and deep gorges have been spanned by substantial bridges, mountain ranges have been pierced by tunnels, and an excellent roadbed has been constructed.

That portion of the line extending from the Missouri River to Seattle, about 1 376 miles in length, was built within a period of three years.

Much credit is due the engineers connected therewith for the satisfactory manner in which the many perplexing problems have been solved.

It is not my purpose, however, to attempt the enumeration of all the great engineering works of the present day, even in our own country. It is gratifying to note that the members of this Society are conspicuous in them all.

Originally, the Constitution required the President, in his address, to give a review of the progress of engineering during the preceding years, but the development was so rapid and the field became so wide that the rule was amended so as to require merely an address.

From what has preceded, I am sure you will sympathize with me in my vain efforts to find a subject, relating to our well-being as a Profession and as a Society, that has not already been ably and exhaustively treated by my predecessors, and you will appreciate the necessity and propriety of limiting my remarks largely to the Society in its present relation to all the members, and the desirability of cultivating a closer personal interest in its management and a pride in its membership.

At the beginning of my membership, the Society had about 600 members, and during that year (1880) the total increase was 10, while at the present time the total membership is 6 550, the increase during 1911 having been 529.

In this gain the three Pacific Coast States are conspicuous with a present membership of 650, or about one-tenth of the total membership.

California, with its 409 members, ranks third in point of numbers, being only exceeded by New York and Pennsylvania, and during the year 1911 it had the largest increase in membership of any State except New York.

The State of Washington has 135 members. The States west of the Mississippi have 1 534 members, while the State of New York alone has 1 733 members, and there are 1 388 resident members living within fifty miles of the post office of New York City.

The preponderance of numbers in the Far East doubtless accounts for the preponderance of influence and interest in the affairs of the Society which is sometimes credited to them.

Members of our Society are also to be found in 39 foreign countries. So it will be seen that it has grown in influence as well as numbers, and still continues to grow at a rapid rate.

It has been said that our Society is too conservative, and as a result new societies have been formed, which could have been satisfactorily provided for within our own organization.

There seems also to be a disposition to avoid participation in the discussion of public questions, even when closely related to the work of the Profession. When Congressional Committees call on the Society for advice with regard to pending legislation, involving questions relating to engineering, it would seem to be a proper function of the Society to render such aid as may be practicable.

In fact, it might be well, under proper conditions, to go even farther, and use the influence the Society may have to mould public opinion along lines free from local or political bias, when our public works are the subject of discussion.

These and other problems bearing on the aims of the Society, have often been discussed, and will continue to receive consideration, with a view to increasing its usefulness.

The influence of our Society will always depend on the character of its membership, and to this end every one connected with the Society should, as far as may be practicable, take a personal interest in the admission of its members. The Board of Direction must rely on the members for the information needed to judge of the professional and personal fitness of an applicant for admission.

If this duty, common to all members, is conscientiously performed, the simple fact of being a member of the Society will be of far greater value than any license that could possibly be given by legal enactment.

The only good purpose to be served by a system of license, to authorize an engineer to practice his profession, would be to protect the employing public from incompetent men.

Membership in the Society, if carefully guarded, would soon be accepted as a far better guarantee of fitness than any license system that could be devised, and the expense, to both the public and the engineer under such system, which would be a considerable sum, would be wholly unnecessary, as Membership would at once be a certificate of qualifications to design and direct engineering work successfully. The Society itself should be the first to condemn an unworthy member, but should be equally prompt in defending a member who is unjustly accused of wrong doing.

As a matter of fact, the Society is already regarded by the public so highly that its members are looked on with special favor by the Courts when expert testimony is required, by the Government when seeking for capable men for service on public works, and by municipalities where men of integrity and ability are looked for to fill positions of trust relating to the engineering side of city government.

In a speech at the Society House recently a leading politician of New York announced that they have come to realize that the interests of the city are best served by appointing engineers to fill the important offices which have charge of the physical welfare of the city in general.

This is true of many of our cities, and the Profession is steadily growing in public favor; loyalty to the Society on the part of all its members wherever they may be located will greatly stimulate this growth.

No matter how remote you may be from the House of the Society, do not for a moment harbor the feeling that nothing is gained by membership therein, except the copies of the *Transactions* and the privilege of hanging up a certificate of membership and wearing the Society badge.

The Society belongs to you, and a personal interest in all its affairs will make a better Society as well as a better member. The Board of Direction, which represents you, will welcome such interest, to the end that the usefulness of the Society may be broadened.

It is said by some that the Society is not truly National because it has its house in New York where are held the Annual Meetings and the meetings where papers are read.

That the proper place for the home of the Society is in the Metropolis of our country can hardly be questioned. In no other city could there be found such a large resident membership, which equals one-fifth of the total membership. The excess dues charged to such members cover nearly one-third of the entire cost of maintaining the Society. So, from a purely business point of view, the situation must be regarded as satisfactory.

If ways and means can be devised to better the practice as to meetings, it lies in the power of the members to have it done.

The speaker has given some study to this question, and finds a possible remedy in more frequent conventions or meetings in different localities readily accessible to a considerable number of members.

The distances are so great that a convention is necessarily made up almost wholly of members comparatively near at hand. This is more particularly true since the system of free transportation, formerly generously accorded to us, has been abolished. Although the Society has greatly increased in numbers, the attendance at conventions has decreased, due chiefly to the cost of transportation, which was formerly free.

Take, for example, the convention this year on the Pacific Coast. Comparatively few members can devote the time and expense involved in a trip from near the Atlantic seaboard, and if another convention were to be held in the East a month later, there would doubtless be a larger attendance than we have here to-day.

More local associations, for reading and discussing papers, would also add materially to the interest of the members in the Society; but, even with these, the acquaintance and affiliation of members from different localities would not be realized, and this is one of the chief advantages of general gatherings, such as our Conventions and Annual Meetings, that can hardly be reached in any other way.

The proper solution of this question is worthy of your best thoughts, and when you have reached satisfactory conclusions, give the Board of Direction the benefit thereof.

The selection of a suitable place for the Annual Convention has often been quite embarrassing, as many localities are generally named in response to requests for suggestions from members as to the most satisfactory place for such meeting.

Many places are usually suggested, and it is rarely that any one place receives such a majority of votes as to indicate a decided preference, and the matter, after all, must be decided by the Board of Direction.

It not infrequently happens that the members in a given district feel slighted because their invitation is not accepted, and feel that favoritism is shown in making the selection. Whether there is just ground for such belief or not, a remedy lies in the hands of the members.

Suppose we let the general locality of the Annual Convention be fixed in the several districts in certain proper numerical order, and let the members of each district determine the specific locality therein where the meeting shall be held. The location will then be automatic to a considerable extent, and no one need feel slighted.

Whenever special reasons of sufficient moment should make it desirable to waive the regular order of progression, it seems hardly probable that serious objection thereto would be urged. A friendly rivalry between the different districts might be developed by such method of selection, which might prove to be highly beneficial.

These suggestions are made in the hope of stimulating the personal interest of all members in the management of the affairs of the Society, not only as a privilege, but as a duty.

It is related of Telford, the greatest engineer of his time, that he offered no encouragement to young men who aspired to enter the Profession of Civil Engineering. On the contrary, he told them there was nothing left to do in Great Britain, as he had already built all the canals, roads, and harbors that the country required.

He died seventy-eight years ago, and the intervening years have witnessed the planning and construction of far more and greater works than were even dreamed of in his day and generation, and the end is not yet.

In our own country, the developments in many lines have been so rapid, and projects of great magnitude have been conceived, planned, and completed so rapidly, that there would seem to be an end thereto; but, day by day, come new projects. Existing works have become obsolete, or the capacity has become too limited to satisfy the needs of the rapid increase in population, hence we must tear down the old and build larger and better. These changes are so rapid that it requires looking well into the future to determine, as best we can, how to build the works of to-day so as to meet the requirements of to-morrow.

The locks at Sault Ste. Marie which serve the traffic of the Great Lakes have been rebuilt and enlarged four times to meet the increased demands of the great volume of traffic which passes through them.

Had the Panama Canal been built on the lines laid down by De Lesseps, it would have been obsolete before it could have been completed, and reconstruction on a larger scale would have been imperative.

Much of our own work in the past has been done with little reference to permanency, but rather to meet a present emergency, and such work must all be reconstructed or better work substituted. The sharp curves and steep grades of our railways must be eliminated; the bridges and tracks that were ample for our traffic of light locomotives and 20-ton cars of past years, must give way to more substantial structures that will carry the giant engines of to-day hauling long trains with car loads of 50 tons.

The channels and harbors with 20-ft. depth, which answered the purpose a few years ago, must be developed to depths of from 35 to 40 ft.

The cities have grown from a few thousand souls to hundreds of thousands and even millions, and the water supply, the sewers, the streets, and all that is required for the health and comfort of the people must keep pace therewith. New cities are also springing up, and these must be equipped with modern appliances.

Our Government has yet to adopt a definite policy as to the improvement and flood control of our numerous waterways. While considerable work has been done on the larger streams, it is trifling in amount as compared with what must yet be done in the development of streams for navigation, in the conservation and use of the waters for power, in the control of floods, and in the drainage of wet lands.

Then, too, a large portion of our territory is yet undeveloped, so it is apparent that the demands for engineers will continue to increase, and the rewards for their labors will continue to grow.

It seems evident that there is still far more work for the Civil Engineer to do than has yet been accomplished, and a bright and busy future is assured for many years to come.

In this development our Society has important obligations to fulfill to the Profession and to the public. It should fix and require such moral and professional standards that membership will at once be recognized as a guarantee of integrity, honor, and ability, so that its members may merit and receive the fullest respect, esteem, and confidence of their fellow-men.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 1237

HOW TO BUILD A STONE JETTY ON A SAND BOTTOM IN THE OPEN SEA.*

By Henry C. Ripley, M. Am. Soc. C. E.

With Discussion by Messrs. L. M. Haupt, Frederic V. Abbot, John Taylor, Morton L. Tower, Howard J. Cole, J. Francis Le Baron, L. J. Le Conte, and Henry C. Ripley.

Introduction.

Within the last 25 years a method of jetty construction has been developed in the United States, which, for stability and economy, has never before been approached, as far as the writer is aware. As no complete description of this method has ever been published, it is the writer's purpose to discuss it in detail, in accordance with its most recent development, and give the reason for each detail, so that those who are unfamiliar with work of this class may appreciate its utility. The order of construction by which bar advance may be minimized, if not wholly prevented, will also be described.

Explanation and Definitions.

A jetty consists of the four distinct elements: the foundation, the core, the side-blocks, and the crest or cap-blocks. That portion above the foundation, taken together, is the superstructure. The foundation is a thin layer of stone—usually about 3 ft. thick—of the full width of the jetty; that part projecting beyond the superstructure is called the apron. The core is the interior portion of the superstructure, and

* Presented at the meeting of January 3d, 1912.

the crest-blocks and side-blocks are the large blocks of stone covering the crest and sides of the core. A section of the superstructure will have the form of a trapezoid. The slopes of the sides will be such as the stone assumes in placing, and will vary according to the degree of exposure to wave action and the size of stone used. Generally, however, they may be kept within the limits of one horizontal to one vertical (1:1) and two horizontal to one vertical (2:1). The slopes will generally be unequal, because one side will usually have greater exposure to the waves than the other. The top width will vary with the degree of exposure to wave action, but it should not be less than 10, nor more than 20, ft. If the latter figure is exceeded, it will indicate either improper construction or unusual exposure to wave action. Naturally, the width at the bottom will depend on that at the top, the side slopes, and the height of the work. The volume of material, and hence the cost of the completed work, will vary with the top width, the side slopes, and the height; therefore it is important to prevent these dimensions from assuming unnecessary proportions. To be sure, the height of the jetty will depend on the depth of the water where it is located, and this cannot be controlled. However, by preventing scour in advance of the work, under-scour by cross-currents through or under it, and the undermining of the sides by wave action or longitudinal scour by current action, much may be done to curtail the height which might be necessary were these details neglected.

Before proceeding further it will be useful, and necessary to a clear understanding, to define the different classes of stone entering into the construction. These are as follows:

Small Rip-Rap.—This consists of small irregular pieces of stone, each weighing from 10 to 100 lb., which may be handled by one man; it is sometimes called "one-man stone."

Spalls or Chips.—These are pieces of stone smaller than the small rip-rap, and are usually handled with a shovel.

Large Rip-Rap.—This consists of irregular pieces weighing from 100 to 1 500 lb., or even more, of almost any shape which the quarry will produce, although very thin and flat pieces are objectionable because they tend to break into pieces in placing and it is impossible to make a good bond with them.

Large Blocks.—These consist of more or less regular pieces of stone, weighing from 1 to 10 tons, or more.

Method of Construction.

The foundation must be constructed first. It is made of small rip-rap, large rip-rap, and spalls. There must be enough large rip-rap to hold the small rip-rap in place against wave action, and enough spalls to fill the voids in the rip-rap and prevent the undermining effects of cross-currents by working through and under the mass. It must also be of sufficient thickness to prevent the breaking waves from jetting through the mass, and washing out the sand from under it. A liberal allowance of spalls will always assure a secure foundation against current and wave action through the mass. Where wave action is very great, it is necessary to place the small rip-rap first and cover it with the large rip-rap, in order to hold it in place, the voids in the latter being filled with small rip-rap. Where wave action is not great, the large rip-rap may be dispensed with and the thickness reduced.

In fixing the width of the foundation, two things must be considered, namely, the width of the base of the superstructure and the width of the apron. The first will depend on the height and on other conditions which will be discussed later. The width of the apron must be such as to provide for any undermining and settlement due to overfall during or after the completion of the whole work, and for that due to scour, principally along the channel face. The probable depth of scour can generally be predicted with a considerable degree of certainty, and, by watching the results as the work progresses, the amount of deepening caused by overfall may be closely approximated for ordinary conditions. For those extraordinary conditions which are due to great and unusual storms, the results are more problematical. However, the direction from which great storms come is generally known, and, therefore, the side of the jetty on which the overfall effect will occur. When this happens to be on the channel side, the provision for channel scour will have provided sufficiently for any action due to overfall which is likely to occur. Where there are two jetties, the one suffering from overfall on the outside will be in the lee of the other jetty, and the effect will be thereby diminished. It will generally be found, therefore, that the maximum overfall effect will be developed during construction, and if this be ascertained and fully provided for, no great fears need be entertained for the stability of the work after it shall have been completed.

In determining the necessary width of the apron, in addition to the provisions for overfall and current scour, a width of from 4 to 6 ft. should be provided for a berm at the base of the superstructure. This berm will furnish a secure footing for the large blocks covering the sides of the superstructure, and will prevent them, when rolling down, from continuing on down the slope of the apron. It will also serve as additional security against undermining the superstructure in case of excessive scour from any cause. The slope assumed by the apron will depend on the degree of exposure to wave action; hence, the deeper the water in which it is placed the steeper will be the slope up to the angle of repose, which may be taken at 45 degrees. Generally, this slope may be taken at from 1 : 1 to 1 : 1½. It will always be steeper than the adjacent side slope of the superstructure, unless the construction be made in absolutely quiet water.

In designing the foundation for a jetty, it will be necessary to determine its width at every point, as this will vary in accordance with the conditions mentioned. As an example, let it be required to determine the width of the foundation at a point where the water is 12 ft. deep, and let it be assumed that the top of the jetty is to be 3 ft. above the water surface, that the top width is to be 15 ft., that the side slope on the channel side will be 1 : 1¼, and on the sea side 1 : 1½. Let the foundation be 3 ft. thick; suppose the scour on the channel side to be down to 25 ft. and on the sea side to 16 ft., and assume a slope of 1 : 1 for the apron on the channel side and of 1 : 1¼ on the sea side, with a berm of 5 ft. on each side. The height of the superstructure will then be 12 ft., and its bottom width will be $15 + 15 + 18 = 48$ ft. The width of the apron on the channel side will be $5 + 13 = 18$ ft., and, on the sea side, it will be $5 + 5 = 10$ ft. The total width of the foundation at this point will then be $48 + 18 + 10 = 76$ ft.

When properly constructed, the foundation serves two important purposes: It secures the superstructure against settlement from undermining, and it protects the bottom, on which the jetty is to rest, from scour in advance and during the building of the superstructure. When once commenced, the construction of the foundation should be pushed as rapidly as practicable, for any delay will permit scour at the unfinished end, which must afterward be filled in with stone, and any deficiency in the full width may permit the edges to drop down, and additional stone will be needed to restore the loss in height. Any temporary deficiency in width which may be necessary in the course of construction, therefore, should be confined to the apron, where a moderate settlement will entail no loss, that is, beyond the base of the superstructure and berm.

Superstructure.

On the completion of the foundation, the superstructure may be commenced, or, in some cases, it may be commenced before the entire completion of the foundation. This will be when its building-up will not cause an increase in the cross-currents in advance of the foundation, and where the concentration of the outward flow is not likely to cause advance scour.

The first portion to be constructed is the core, which is composed of large and small rip-rap and spalls. The small rip-rap and spalls secure tightness, and the large rip-rap prevents the small rip-rap from spreading out and flattening the slope on account of wave action. The flattening of the slope means greater volume, and, therefore, increases the cost of the work.

Where the water is deep, or where there is little wave action, the core may be commenced by depositing the stone along the axis of the superstructure, using large and small rip-rap and spalls in such proportions that the small rip-rap will fill the voids in the large rip-rap, and the spalls will fill those in the small rip-rap. As the core of the wall approaches the elevation of low tide, or, say, within about 3 ft. of that plane, this form of construction will not suffice to prevent the stone from spreading out and flattening the side slope. When this stage is reached, the sides of the work must be covered with large rip-rap which must be brought up to a height above the general level of the work in two ridges parallel with the axis of the superstructure. The space between these two ridges will then be filled with small rip-rap and spalls, and, in some cases, it may be necessary to use also some large rip-rap, in order to prevent the smaller stones from being washed out by the waves. Then the ridges must be raised again, and the space between them filled as before. This process must be continued until the proper height of the core is reached, the top having been completely covered with large rip-rap. This height should be such that, with the addition of the crest-blocks, the work will be brought to the full height required for the superstructure after consolidation has taken place.

The allowance for consolidation is largely a matter of judgment. It will depend on the height of the wall, the character of the stone used, the intensity of wave action during construction, the number of unfilled voids in the mass, and the pounding it is likely to receive from storm waves subsequent to completion. The pounding of waves on a mass of stone sets up a vibration which causes the points of contact of the different pieces to rub on each other, and this, by wearing away the stone, permits the mass to consolidate. When this process has been continued for a certain length of time, however, this vibration becomes ineffectual in reducing the bulk, the structure practically ceases to shrink, and the mass becomes stable. Moderate wave action on the work during construction is not an unmixed evil, as it helps the stones to pack together and consolidate, and thus the subsequent shrinkage is reduced. The more nearly water-tight the jetty is made, the more efficient it will be, because there will be less leakage and waste of the water which its purpose is to constrain, and because of the smaller quantity of sand which can pass through it to the injury of the channel.

Having regard for these considerations, the allowance for consolidation will vary from 6 in. to 2 ft.; but, where the work is subjected to the usual wave action during construction, the shrinkage will rarely exceed 1 ft.

As soon and as fast as the core is finished, the side- and crest-blocks should be placed, for otherwise a severe storm would damage the uncompleted work. The side-blocks should be placed one at a time, and allowed to roll down until the sides are completely covered from the bottom up to about the elevation of low tide. Above this elevation each block must be carefully placed so as to fit as closely together as practicable. As soon as the top is reached, the surface of the core must be leveled up with small rip-rap, and, if there has been any loss in height by the washing down of the crown, the proper elevation must be restored before capping. The cap-blocks will then be selected, using only those pieces which will give the proper height to the crown, and these should be placed so as to leave as little of the sides as practicable exposed to the waves. The side-blocks should be disposed in relation to the cap-blocks so that the waves will glide over the latter rather than strike with full force against their sides. When these blocks are properly placed, the surface of the core is completely covered, and when the inevitable shrinkage due to consolidation takes place, these blocks wedge themselves together in such a way that the heaviest storm waves cannot dislodge them.

Where one side of the structure has greater exposure to wave action than the other, the larger blocks will be placed on that side, and, as the greatest force of the waves is exerted on that portion of the work above the plane of low tide, much care must be taken in building this portion. This is greatly facilitated, however, by the fact that it is always exposed to view at low water, and there can be no uncertainty as to the position of each and every block.

Stone.

The character of the stone to be used in a work of this nature is of considerable importance. Hardness and weight are prime requisites, especially for the side- and crest-blocks, and, for these, granite, gneiss, limestone, sandstone, or other stone of considerable specific gravity, should be used. The value of a stone to resist wave action varies directly with its weight under water and inversely with its surface exposure. Its value depends on its specific gravity, and to an extent which is not always fully realized. Suppose two stones, *A* and *B*, each having the same weight (say, 1 metric ton or 1 000 kg. in air), *A* having a specific gravity of 2.1, and *B* a specific gravity of 2.7. Then *A* will have a weight, under salt water, of 496 kg., and will displace 504 kg. of water, while *B* will have a weight of 608 kg., under the same conditions, and will displace 392 kg. of water. Then *B* will have a weight under water $22\frac{1}{2}\%$ greater than *A*, a volume 22% less, and a surface exposure to wave action $15\frac{1}{2}\%$ less than *A*. Therefore, it will have 38% more stability or power to resist wave action than *A*, though its weight in air is exactly the same.

For the foundation and core there is not the same economy in the use of heavy stone, but the large rip-rap (the main purpose of which is to resist wave action), if of high specific gravity, will enable the core to be built with a steeper side slope, and, therefore, will conduce to economy in the quantity of stone required for a given work. For this reason a limestone is generally preferred to a sandstone of the same grade as to hardness, because of its greater specific gravity. For a foundation in deep water and for the interior of a core, a stone of less hardness is not altogether objectionable, because the chips which result from handling help to reduce voids and consolidate the work sooner. In some cases, the use of a stone of small specific gravity for the foundation will give satisfactory results, with some economy in tonnage.

Order of Construction.

When jetties are designed to control the flow of water across a sand-bar, the order of construction is of vital importance. The order in which the material should be put into the work has already been given under "Method of Construction." It is here proposed to consider whether the work should be built from the shore outward or from the outer end shoreward.

Suppose that two parallel jetties are to be constructed at the mouth of a river, or at the entrance of a tidal harbor, to extend from the shore out across the bar; and suppose that the foundation of each has already been constructed. The position of the bar is determined by the equilibrium of the forces, one set of which tends to push it seaward, while another tends to push it back toward the gorge. When these forces are equal, the distance of the bar from the gorge is constant.

If the superstructure is commenced at the shore ends and extended toward the outer ends, it will have the effect of advancing the gorge toward the bar, with the resulting advance of the latter. If the work progresses rapidly, the jetties may overtake the bar advance and get across it, but such a result will always be at the cost of a considerable extension of the jetties beyond that originally required. If the work of construction progresses slowly, the jetties may never reach and cross the bar, and the expenditure may become so great that the project may be abandoned without accomplishing the purpose for which it was designed.

If, however, the construction of the superstructure be commenced at the outer end of the jetties and continued shoreward, there can be no gradual advance of the gorge toward the bar with the consequent bar advance. On the contrary, as the completed work is extended, the waterway will suffer a contraction, the old gorge will disappear, and a new one will be established at the outer end of the jetties at a point beyond the crest of the bar. With the increasing contraction of the waterway, there will be an increased current, both between the jetties and laterally through the gaps between the completed work and the shore on either side of the channel; but, as the foundation work of the jetties will prevent enlargement laterally, deepening of the channelway between the jetties will be inaugurated, and, as the greatest tendency to scour will be at the outer end of the jetties, deepening will commence at this point and extend backward with the advance of the completed work shoreward. The material thus eroded from the channelway will be carried beyond the outer ends of the jetties, where it will be either swept to one side by the littoral current or deposited in the deep water farther out.

Another advantage in working from the outer end toward shore is the facility afforded in construction. The work will thus be carried on in the lee of the finished structure, and in this way the number of possible working days is considerably increased. Where the work is being done with a floating plant, this increase in available working days may amount to from 50 to 100%, with a corresponding saving in operation expense. Even where the work is being done by using a trestle, the construction is greatly facilitated by being in the lee of the finished work, and much time may be utilized which otherwise would be lost.

For a single curved jetty, a detached breakwater, or a training wall, where waves and current are encountered, the same general method and order of construction should be followed.

Where the conditions are favorable for the construction of a trestle along the site of the work and the cars of stone are run directly on it, a most convenient method of construction is furnished. It enables the work to be carried on during weather which might be so rough that it would prevent the use of a floating equipment. It also enhances to a considerable extent the rate of progress. The floating equipment, however, is very convenient for placing the foundation, and, on a large work, a combination of the two systems may prove advantageous. It is beyond the province of this paper, however, to enter into the subject of the method of handling the stone. Either method will give entirely satisfactory results.

DISCUSSION

L. M. Haupt, M. Am. Soc. C. E. (by letter).—It would seem that, after many centuries of practice in building structures of this important class, there should be little to be learned, and yet the author states in his opening paragraph that the art has been developed to a high stage only within the last 25 years, in the United States, and he proceeds to show the most economical order and proper dimensions and materials for the construction of works exposed to the violence of the sea. In view of the large deterioration, and the tentative methods which have been followed, it would appear that his analysis of the requirements is timely, and may serve a good purpose for the tyro who may be willing to be guided by the experience of his predecessors. Mr. Haupt.

In ancient days, when ships were propelled by the winds, or triremes were driven by slaves, the depth of water over bars was of comparatively little importance, but, with the evolution of the ocean liner, requiring a clearance of 40 ft., the problem has assumed a world-wide significance; and jetties, as well as moles and breakwaters, have become essential as aids to navigation and elements for the salvation of life and property. The cost of these extensive structures, however, may be prohibitive, and, if numerous, may become a serious drain on the exchequer of a country, especially where the commercial development is in its infancy, as in many undeveloped colonies requiring aid from the mother country. The policy of appropriating small sums for partial constructions, with intervening suspension of works, leaving them unprotected and exposed to violent storms, necessarily adds greatly to the cost and reduces the possibility of securing the channels desired. This has made it necessary to proceed tentatively, building a little at a time, and at first with submerged profiles, then to low water, followed by mid-tide, and finally to above high water, to secure the desired control. In cross-section, also, a similar progressive policy has been pursued, and the hearting has been placed on mattresses of brush laden with small stones and covered with larger rubble or rip-rap, with but little attention to the relation of the foundation-sill to the superstructure or the protection of the exposed ends of the work, subjected to violent wave or current action, with the result that the reaction of the jetty at its end invariably scours out the bottom, and makes it necessary to supply rock for the sand *in situ,* at much greater cost.

Efforts to reduce the cost by depositing clay or dredged material in the hearting, to render it more impermeable, have also resulted in disaster and added expense, and by adopting too great a height and width above high water, greater pressure and resistance have been offered to the maritime forces, with consequent greater deterioration.

Mr. Haupt.

All these elements the author has carefully considered, and the conscientious observance of his suggestions should result in great economy because of the more rapid and stable execution of the works. It does not seem to be necessary to point to specific instances of failures as indices to the conclusions reached, for the author's reputation as an experienced engineer, for many years in the service of the United States on the Gulf of Mexico, as a Member of the Board to design the great sea-wall at Galveston, Tex., and as Consulting Engineer for some of the principal harbors in South America, is a sufficient warrant of his competency to formulate the most desirable method of procedure for structures of this class; but, the works of the Phœnicians some 3 000 years ago, when they established harbors and ports on the coasts of the Great Sea (Mediterranean), especially at Tyre and Sidon, which are celebrated to this day,* indicates that there have been others skilled in the art of building sea-walls and jetties.

The insular Tyre was about one-third of a mile from the shore, with which it was connected by Alexander the Great, when besieged about 383 B. C. Strabo states that the walls were 150 ft. in height and correspondingly thick, and were built of massive blocks of stone bedded in mortar. Nebuchadnezzar had failed to take it after a siege of 13 years, but Alexander tore down the portion on the main land, and with its stones built the mole 200 ft. wide to the island. Cresy says:

"A violent storm of wind arose, and destroyed a portion of the work, * * * [no unusual experience]. This was speedily repaired, by causing large trees, cut down in the mountains, to be thrown in, with their branches entire, on which was heaped a quantity of earth, to render it strong enough to resist the violence of the sea."

Over this he advanced his battering rams, and the siege was on in earnest. The insular Tyre, destroyed by Alexander, is now a "place for the spreading of nets in the midst of the sea," as Ezekiel prophesied. The mole which the conqueror raised was washed away by a storm, and thus the peninsular Tyre was destroyed. The 9th Edition of the Britannica states: "The mole which he constructed * * * has been widened by deposits of sand, so that the ancient island is now connected with the mainland by a tongue of land a quarter of a mile broad."

Mr. Abbot.

FREDERIC V. ABBOT,† M. AM. SOC. C. E. (by letter).—This paper is peculiarly interesting to the writer because the procedure recommended therein is almost exactly that adopted at Charleston, S. C., in the years between 1884 and 1890. At that locality log mattresses formed the sub-foundation of the greater part of the jetties, but an

* Plans of these may be found in Edward Cresy's "Encyclopædia of Engineering," 1847

† Colonel, Corps of Engineers, United States Army.

apron of spalls and small stone, in lieu of logs, was adopted in some parts of the work, and proved thoroughly reliable and easy to construct; it was furthermore exempt from injury by the teredo. The original foundation mattresses, however, where promptly covered with 2 ft. of spalls, have given no signs of deterioration in the past 25 years. Mr. Abbot.

The writer would emphasize the desirability of incorporating some large 1-ton to 7-ton blocks in the hearting. It adds stability to the core during construction, and, after completion, a tearing away of parts of the outer skin of heavy blocks by storms does not produce a general disintegration of the whole structure near the injured parts due to the washing out of long stretches of the core.

One advantage of such incorporation is that this use of large stone makes it possible for the engineer to conduct the work so as to utilize at all times the whole output of the quarry; in this way the costs at that end of the line are much reduced.

A jetty with a spall foundation and a superstructure formed simply by the deposit of regular quarry run along the axis, or a little to the seaward of the axis, is in some situations cheaper than one built to predetermined cross-sections, because the action of storms will show automatically where the wave action is a maximum, and where it is not so great. Points at which the waves knock down the original superstructure are evidently the loci of specially severe attack, and by raising these portions a second, or third, or fourth time, or increasing the size of the stones used, all material is placed just where it is needed, and none is wasted in building up a theoretical cross-section which may be stronger than is required. In any jetty it is well if possible to delay the final covering with the largest blocks until there has been time enough for ocean waves to develop weak spots.

At Charleston the foundation was built out from the shore, but the outer ends of the superstructure were completed first and were extended toward the shore until surveys showed that sufficient concentration of ebb flow had been secured to give the depth desired in the jetty channel. To avoid the formation of a sinuous channel—hard to navigate—scour was directed by dredging, but of late years neither dredging nor repairs to the jetties have been needed, and the originally projected depth has been somewhat exceeded. Scour has now nearly ceased, the cost of maintenance has been practically nothing, and the dredges have been utilized at other harbors.

To provide greater depth at Charleston it seems probable that simple dredging is all that is necessary, the principle being well established that jetties will maintain a greater depth than their unaided action will originally create. It is pleasant to the writer to read a paper which accords so closely with his own ideas.

Mr. Taylor.

John Taylor, Assoc. M. Am. Soc. C. E. (by letter).—Mr. Ripley's interesting paper presumably includes within its scope the design as well as the construction of a stone jetty, as he suggests certain dimensions for the cross-section, etc. Further information as to the conditions and exposure under which such designs are used would increase the value of the paper to other engineers engaged on the design of structures of this class, and it is hoped that the author will kindly supply this.

It is to be presumed that he does not intend such steep slopes for use on the sea face under all circumstances, whether adverse or otherwise. The widths he states to be subject to variation to suit the exposure and other local conditions. The writer has personal knowledge of sites on which breakwaters of the rubble mound type have been constructed where, entirely apart from the question of width, such slopes on the sea face would be cut into and breaches would be made in the superstructure in a very short time if stone of only reasonable size and such as is generally available were used. The larger the blocks of stone the steeper, of course, will be the slope at which the rubble may be expected to stand. In some European ports in exposed sites, where very large concrete blocks are used as a wave breaker on the sea face of rubble mounds, they stand at a comparatively steep slope, say, $1:1\frac{1}{4}$. In others, where more moderately sized but still large rubble stone, weighing 1 ton and upward, have been used for this purpose, the slopes have been dragged down between tides and to varying levels below low tide to final slopes as flat as $1:10$. This, of course, depends on the exposure and the material used, and it appears to the writer to be extremely difficult to lay down any hard and fast rule.

A sea water-wave of a given height develops the same energy, and consequently the same destructive and erosive power, in any part of the world, but its effect depends on the manner in which that energy is expended, that is, whether or not it is in shallow water, and also on the contour of the artificial obstacle placed in its way, which may convert its normal motion into a particularly violent forward and vertical one. This introduces the question of the comparative value of vertical walls, battered walls, walls with a submerged talus, mounds with long gradual slopes approaching to the contour of a natural beach, or those with a medium slope of, say, $1:2$, and their effects on waves and consequently on themselves. There are also the many freak designs, introducing berms, ramps, and even sections of ellipses or parabolas on the sea face. These are founded on some assumption of the designer and, on the face of them, are inapplicable in positions where there is any considerable tidal range, as they generally assume a nearly uniform water level as a starting point on which the theory

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is based. Undoubtedly, the nearer the form of the work approaches to the vertical in deep water the less is the wave energy expended in a destructive form against it. The trouble with a rubble mound is to maintain it with the steep slope after construction. With such a slope a wave does not have time to acquire a violent forward movement, but, against this, there is the natural tendency of the rubble under erosion to assume a lower angle of repose.

The word, jetty, is evidently applied by Mr. Ripley to the semi-breakwater-training walls for regulation and protection purposes at the entrances of river and lagoon harbors, such as have been built at numerous points on the United States coast line. They are generally placed approximately at right angles to the general coast line, and parallel to the channel center line. The inner and less exposed sections of these are often subject to only moderate seas running across sand flats. The outer ends are often much exposed, and here the chief trouble in their maintenance is found. Perhaps the author will state the degree of exposure and wave height which such a structure as he typifies would withstand successfully with moderate maintenance.

Such designs have undoubtedly been successful and ecomomical under fairly moderate conditions. The writer does not think, however, that they would survive in the more exposed sites on the Atlantic and Pacific Coasts without heavy maintenance charges, unless wider cross-sections, very heavy material, and much flatter slopes were used on the sea face. In a paper by Morton L. Tower, M. Am. Soc. C. E.,* it is stated that:

"for 800 lin. ft. the outer end of the Coos Bay Jetty was built three times to 24 ft. above low water and beaten down at the extreme end to 20 ft. below low water, or only a few feet above the surrounding sand."

It is one thing to design such a structure with steep slopes, and another to keep it so for any length of time; for repairs often reach such proportions that many of these works can only be looked on as still under construction for long periods after they are nominally completed according to the design. The slopes require continual additions in order to replace dislodged blocks until a final permanent condition is attained. Of course, this does not apply in all cases, but in very many it has proved to be absolutely necessary. Exposure in a rubble mound, as affecting the life of the structure, is not necessarily defined or limited by the direction of the work relative to the direction of prevalent storms and the general shore line, as the scouring action of waves from occasional storms, striking the work, head on or at an acute angle, will sometimes draw down the rubble slope as effectively as would waves attacking the structure more nearly at right angles.

* *Transactions*, Am. Soc. C. E., Vol. LXXI, p. 354.

Mr. Taylor. The secret of success in a breakwater with steep slopes, such as described by the author, is undoubtedly the proper protection of the core by an outer covering of very heavy blocks on the superstructure and slopes. The weak point, however, is generally the foundation toe of the slope blocks, which receives the thrust of the wedging of the blocks caused by the settlement of the supporting core, due to the dead weight and the consolidation by wave impact. The wedging effect is the source of strength in the superstructure in such a design, if the toe is made secure.

The writer has had experience in the construction of 9 000 lin. ft., and the maintenance of about as much more, of rubble mound breakwaters, at the Portland Naval Harbor in Great Britain, running out into water 60 ft. deep at low tide. The different parts of the work were exposed to varying heights of seas due to different relative exposures, and several cross-sections were adopted to meet these conditions. Waves wheeling from the open Atlantic, up to 25 ft. in height at a moderate estimate, were projected against the most exposed parts, and here the sea slopes were flattest, averaging about 1:6 from high tide down to about 20 ft. below low tide. Other less exposed portions were steeper, but none was steeper than 1:2½, even in the most sheltered parts, where 15-ft. waves were felt. A short temporary section, having a slope of 1:1¼ on the sea face failed to stand at the latter site. The upper part of the work and the slopes down to low tide were covered by heavy stone "pitching," about 3 ft. thick and up to 10 tons in weight. The material is limestone from the low-grade beds in the quarries, has a specific gravity of about 2.45, and is very tough and durable. Below low tide the slope was deposited at random from dumping barges. At low tide a heavy toe of large, loose rubble was placed in order to protect the lower end of the "pitching." In one part the superstructure, down to low tide, has a slope of 1:2 with a 6-ft. berm of loose rubble as a toe for the pitching, and with the rubble below this level down to the sea bed continued on a slope of 1:2½. With an exposure in this portion giving waves sometimes 20 ft. high, considerable trouble was caused by the loosening of the toe blocks at low tide by wave action and the squeezing out effect due to the wedging of the pitching by the settlement of the core. The only preventive was found to be the addition of large quantities of heavy rubble along the toe, and the flattening of the slope under water to about 1:4 down to about 20 ft. below low tide, so as to provide a solid support for the toe of the cap blocks. In certain cases, of course, sand will sometimes accumulate against the work after its construction in shallow water, and thus provide a natural protection for the toe, but in other cases it may be eroded from alongside the mound and weaken it. The berm at low tide may also in itself be a weakness at certain stages of the tide in a storm, as it tends to trip up waves approaching the work and make them strike the lower cap blocks a heavier blow, while the resultant suction and undertow on the wave recoil tend to remove the loose berm blocks and dislodge the capping. Mr. Taylor.

In such work it is very difficult—and the cost is almost prohibitive —to place successfully and permanently by hand under water a continuation of the cap blocks, down the slope, and the only alternative is a heavy toe of loose rubble, from low tide downward, placed at a stable slope, which will depend on the exposure of the site.

Does the design given by Mr. Ripley ever contemplate the continuation of the hand-laid blocks down the slope below water to the foundation level? In deep water this would place the toe beyond the reach of wave action. On a site which dries at low tide the placing of the lower blocks can generally be easily done.

The trouble with a faulty foundation toe is that the lower blocks will be loosened by wave action and will slide downward and fall out, thus allowing the joints between those higher up the slope to open out and deprive the cap blocks of unity of resistance and the support given to each other by the thrust due to wedging together under settlement. Soon—following the dictum "United we stand, divided we fall"—they are easily loosened and dislodged by the alternate hammer and suction of heavy seas. The open joints also allow the smaller core stones to be eroded, and then the cap blocks fall in and tilt out of position. This general disintegration, once properly started, progresses rapidly, and satisfactory repairs are often very costly. Systematic inspection and repairs after storms, before the damage has had time to advance too far, will do much to avoid this, but it often happens that proper provision, in the way of maintenance, plant, and finances, is not made to permit of this. It is useless to expect a rubble breakwater in an exposed site to survive in good condition without making such provision, and this question of upkeep enters largely into that of the ultimate cost.

The writer has noted portions of structures of this type where the core had settled away from the capping and left it unsupported except by the adjacent blocks, on account of lack of provision for such settlement, and the hasty placing of the capping before the core had sufficient time to consolidate properly. This, of course, is a source of weakness. As regards settlement in such mounds, the writer took measurements of one of the rubble breakwaters previously mentioned at a point where it reached a total height of more than 65 ft. This work had been under construction for 6 years, the material being deposited rather irregularly along it during that period. After the placing of the capping, and during the 2 years following its completion, it settled at a decreasing speed as much as 15 in. At the end of that time, or 8 years after the placing of the first layers of rubble in the foundation, it was still settling at the rate of about 3 in. per

Mr. Taylor. annum. The sea bed was fairly solid, silty mud, overlying boulder clay, which is a very common sea bottom. In a strong current which causes erosion of the sea bed as the work progresses, the common-sense method is to lay a foundation layer of rubble well ahead of the work. This layer would not in itself cause erosion while being placed, and it would effectively prevent that caused by the increased velocity of the currents around the end of the superstructure as it advanced.

The grading of the material in the core by the use of rubble of varying sizes, so as to fill the voids and give greater dead weight per unit of cube, is a vital point in such sea work, and is of course desirable. Small stones in the outer layers, however, are only a source of weakness, as they are easily eroded by waves.

Some rubble breakwaters—in the light of after experience—have apparently been built unnecessarily wide at the water line and consequently also throughout the section. Some of the first ones built were badly damaged, and there was a tendency to run to extremes in the designs. Any section much greater than that necessary to absorb wave impact without allowing the inner slope to be forced out, is a waste of material, though, of course, it is advisable to provide a generous factor of safety. Too narrow a structure will be endangered by being overtopped by waves. If the top level is made too wide, however, the effect on the cost of the work can readily be seen, as the slopes have to be built to take this width, and the cube is thus increased. What this width is, for a given exposure, only experience of works actually built can decide, and Mr. Ripley's paper indicates that this economical width has been to some extent arrived at definitely on works with which he has been connected, given certain known sea conditions.

There are so many rather indefinite factors in the design and construction of such works that it is almost impossible to reduce the resulting data to an easily used formula, and it is a waste of energy to attempt to apply any complex theory. Some of these varying factors at different sites are the specific gravity and the available sizes of the stone, the exposure and frequency of heavy storms, the depth of water, and the tidal range. The designed slope of the cross-section itself will also affect materially the resultant wave action on the work. Although the tidal range at one place is a constant, the varying depths of water, the sea bed contours, and the location of portions of the work may necessitate variations in the cross-section. In northern countries, like Canada, ice may remove the rubble by ice shoves and by exercising a sort of glacial action, when it floats away. In work of this class there is certainly room for the use of that invaluable commodity, common sense, and an earnest study of the forces of Nature to an extent perhaps greater than in any other form of Civil Engineering construction.

Mr. Taylor.

Does Mr. Ripley propose any special strengthening of the seaward end of these jetties, so as to maintain them at a fixed length? In rubble mounds, the exact location of the seaward end is often rather indefinite. In jetties of the type which he describes, the annual cost of maintenance per unit of length would be of interest, if accompanied by some idea of the exposure at the site of the particular work referred to. The choice between the construction of the so-called vertical concrete or masonry type, for instance, and a rubble mound, generally hinges on the difference between the combined first cost and capitalized maintenance charges in both types. Some very cheap and successful examples of the rubble mound have been constructed. The writer has no prejudices in favor of either type, other than those justified by local conditions.

Mr. Tower.

MORTON L. TOWER, M. AM. SOC. C. E. (by letter).—This paper is a concise statement of the functions and methods of construction of jetties as built at the present time. For extensive jetties or breakwaters, as for no other class of engineering works, the construction must needs be made to fit the many existing local conditions and circumstances.

The selection of the principal materials of construction itself is scarcely ever one of choice, being limited to one or two classes of stone; and the cost of transportation is out of practical consideration for the requisite quantity of material from a distant site.

The side slopes given by Mr. Ripley, namely: "1:1¼ on the sea side and 1:1 on the channel side," seem to the writer to be too steep for general application. For elevations lower than 12 ft. below the plane of low water, a slope of 1½ horizontal to 1 vertical can be depended on in almost any exposure; and between that elevation and extreme high water, where the structure is exposed to severe wave stroke, it is believed that a much flatter slope will be found essential unless monolithic or interlocked masonry is used.

If the wall is to be monolithic or interlocked, a nearly vertical side will be more advantageous than a slope, owing to the water or wave cushion formed against such exposures. Structures of the vertical-wall type are only adapted to locations where the approach of the waves is over water sufficiently deep to allow them to pass without being tripped by the bottom, and their nature changed from an oscillating movement to one of translation. For severe locations the writer does not believe the art of jetty or sea wall construction has yet reached such an established stage that the finished side slopes or the weight of individual units can be safely stated.

Another feature of the paper with which the writer does not agree is the method of construction recommended, that is, commencing at the sea end of the proposed work and building shoreward.

Mr. Tower. In the first place, in many locations, especially in northern waters, this is entirely impracticable. Jetties at estuaries are built over long shoals from 1 to 3 or 4 miles in width, measured from the gorge to deep water beyond the obstructing shoal or bar. If the material is to be deposited from barges, the plant cannot be handled in the breaking waves on or near the shoal over which the jetty is most economically built, except during a few days in the fall. If the material is to be deposited from a tramway, no form of trestle yet used for jetty building is sufficiently strong to withstand a winter season of storm waves without a protection of stone reaching practically to low water. Tramways of the finest and strongest wooden piles, with batter piles and ties of wire rope to anchors for lateral stability, have been tried and found wanting.

In the second place, it would not be desirable to construct a jetty from the sea end shoreward, for the following reasons: With but very few exceptions, the volume of water passing into a harbor is as great as that passing out, and the flood tides are as strong as the ebb current. A larger proportion of flood tide will enter a harbor over the shoal water than will pass out by the same route on the ebb current. There are many reasons why this is so, and a short period of observation at any harbor entrance will prove the fact. An examination of the sand of the bar, beaches, and shoals inside the harbor for a considerable distance up stream will prove that the material has been subjected to severe wave action, and that it is very recently from the sea. Considering these facts, the writer believes that in general the greatest improvement at harbor entrances is created, not by scouring out the bar, the material of which naturally settles just beyond the active effect of the jetty current, but by protecting the channel from the inrushing, sand-loaded currents, along the beach. These currents are aided in disturbing and moving the sand by the breaking waves in the shoal water. The material on reaching the gorge is deposited, and is picked up by the ebb current and carried seaward until the force of the ebb is retarded by dissipation in the ocean on the bar.

Considering the foregoing phenomena, it is held that the shore end of the jetties should be constructed first, and that they should be made tight and of ample height to prevent water-borne sand from being washed over the crest. That the scour to be encountered at the end of the work will add to the expense, is well understood, but for coasts of severe exposure there does not seem to be any practical way of lessening it at the present time.

It is expedient, of course, to construct quickly a length of tramway which can be protected with one season's work, and to provide a protecting mattress or apron over the entire distance, which can be built in a season.

However, perhaps all the foregoing points may be termed details, and these must be studied and the methods to be applied determined for each particular location. Mr. Tower.

HOWARD J. COLE, M. AM. SOC. C. E. (by letter).—In view of the fact that the completion of the Panama Canal will encourage the improvement of the harbors on the Pacific Coast, Mr. Ripley's paper is very timely. In fact, Chili has already taken the initiative, and has appointed a Harbor Improvement Commission which is now constructing under contract an extensive navy yard, dry dock, breakwater, and other harbor works at Talcahuano. A contract for the harbor works at San Antonio has recently been awarded, and bids for the construction of the harbor improvements at Valparaiso are now being received. Mr. Cole.

Similar works are in contemplation for the ports of Arica, Iquique, Antofagasta, and Valdivia. Most of these ports will have rubble breakwaters of heavy cross-section, topped with massive concrete blocks, and will be in waters 50 ft. or more in depth.

J. FRANCIS LE BARON, M. AM. SOC. C. E. (by letter).—This somewhat didactical paper gives in small compass some recognized facts and practical information in reference to jetty building in the open sea, but does not seem to present any facts not previously well known and tried by harbor engineers. As the writer cannot agree with some of the conclusions and methods, he will discuss them from his standpoint. Mr. Le Baron.

In the first place, the author's definition of a jetty is very circumscribed. More than a dozen types of construction are used in building jetties in the open sea, some being adaptable at one place and some at another. Probably Mr. Ripley intended his definition to apply only to the particular method of construction which he was using, or had in mind; but, if he advocates for all cases only the fixed plan which he describes, the writer must take issue with him at once, because, if there is any one thing that thirty years' experience in building sea works has taught, it is that no single type can be used for all places. What may be proper for one river mouth may be entirely unsuited for another.

The destructive forces in an "open sea" vary greatly, according to circumstances, the factors being the "fetch," tides, currents, prevailing winds, hurricanes, sea bottom, offshore depths, and sea worms. None of these conditions is the same in all places; therefore, good judgment is required in the selection of the most appropriate type for a given location, and this selection calls for professional knowledge of a high order, and ripe experience. The writer, believing that Mr. Ripley has both, cannot think that he advocates, for all out-

Mr. Le Baron. side work, a jetty of the type he describes, because the first duty of the engineer is to design works most appropriate for each particular case, and this word, "appropriate," includes strength, durability, effectiveness, and economy.

During the past thirty-two years, the writer (either as assistant, chief, or consulting engineer, or contractor) has been connected with jetty work, in open sea or lake, costing more than $8 500 000, and 98% of this has been built or is now building. The types have included rip-rap rock, log-mat and rock, brush mattress and rock, rock and mud-filled cribs, plank and pile jetty, and log and pile jetty; and, on some of the large works, three of these types have been combined in one jetty. On the Western coast jetties more than 5 miles long are now being built according to the designs of the writer, as consulting engineer. These are of composite type, because it is not necessary for the whole length to be as massive and strong as the outer end. Thus, by using a much lighter and cheaper construction near the shore, much money can be saved. The first 3 100 ft. from the shore is simply pile and single-plank bulkhead, to be reinforced by material dredged from the channel. This is followed by 3 800 ft. of double-plank bulkhead, 5 ft. wide from center to center longitudinally, cross-capped, and filled with brush fascines and broken stone. The remainder is built of log mats covered with rip-rap, and with large rock for some distance from the sea end.

For many years it was thought that loose rock drawn down on the floor of the ocean would be swallowed up in the sand, but this is not the case, except in a few locations, and it has long been known that such rock will not sink more than about 1 ft. in the sand. The reason for using log mats, instead of loose rip-rap, for a foundation, is the saving in cost. Log mats can be made and laid in the work for about half as much as it would cost to fill the same space with rip-rap; and, as there are no teredos, and the logs are always wet, they will be practically imperishable. It is estimated that the use of log mats, instead of a rip-rap foundation, will save more than $500 000 on this work, and, even if teredos were present, they would not attack the logs in the foundation course, as the superincumbent weight sinks these logs in the sea bottom and they are also covered with sand and broken shells by the currents and wave wash, under which conditions it is well known that these sea worms cannot work. In most places, also, this process is materially assisted by a thick growth of shell-fish of various kinds, by which the logs and rocks are sometimes completely covered.

The proportions of the jetty detailed by Mr. Ripley are too small for any exposed place. For outside work on the Atlantic and Pacific Coasts, the writer has always used a top width of 40 ft. and side slopes of 2:1, and believes that, in many locations, 4:1 is better for

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the outside. It must be remembered that structures of this kind are built for all time, and are not like bridges or viaducts, many of which are destined to be torn down when the railroad for which they were erected adopts heavier locomotives or changes its location.

The sea ends of jetties are similar to breakwaters, and it is well known that the action of the sea, on most of these important structures, has reduced the outer slope between high and low water to an inclination varying from 3:1 down to 7:1. In 50 ft. of water the writer has used an apron 125 ft. wide, on each side, with a jetty base of 240 ft., making a total width of 490 ft., or about twice as great as Mr. Ripley recommends; and it was not too wide.

Jetties must be strong enough to withstand the great storms to which they will be exposed every few years, or they will be expensive failures. The author dismisses the question as to the effects of such storms in a naïve manner, as "problematical." In proportioning the width of the apron, he estimates that the damage from overfall is caused by the waves only. It has happened in the writer's experience, that where the jetty crossed the deep channel, a tidal overfall of 1 ft. has set up a whirlpool, or boiling action, and has scoured out potholes 50 ft. deep at the base of the jetty before the apron was laid.

Mr. Ripley's plan of laying the foundation and apron, or a large part of it, in advance of the building of any superstructure, is heartily endorsed by the writer, who has always insisted on this in his later works. The construction of the berm is not made clear in the paper, but, on the channel side, it is presumably made necessary by scour in the channel after the completion of the jetty. If log or brush mats are used in the apron (which is by far the best and safest plan), this berm is formed automatically, if the apron when first laid is of sufficient width. The scour, if any, must then necessarily take place on its outer edge. If it does not undermine the apron, it does no harm; if it does, the edge of the mat with its load of stone drops, accommodates itself to the slope, and effectively protects it. In practice, this is found to be what takes place when such mats, loaded with stone, are used as aprons, and it is one of their great advantages over loose rip-rap.

Mr. Ripley's plan of throwing down a loose foundation course of rip-rap and constructing the jetty thereon, from the sea toward the shore, instead of from the shore outward, is contrary to all precedent, and the writer considers it very dangerous. Possibly this might be done at some small river mouth, not much exposed, and where the tidal rise is small, as in the Gulf of Mexico or the Caribbean Sea, but such a method of procedure at the mouth of a river like the Columbia, on the Pacific Coast, or the St. Johns, of Florida, appears to be quixotic in the extreme. At nearly all large river mouths or estuary bars where there is considerable rise and fall of the tide, there is what

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is known as a "swash channel," which runs close to the shore and is generally a small shallow run formed by the flood tide; and this has to be crossed by the jetty.

On Cumberland Sound (Georgia and Florida) the foundation course of the north jetty was laid from shore nearly 4 miles seaward over the bar, all of it, except where it crossed the swash channel, being on a sand bank, awash, or only a fraction of a foot above low water. This foundation course consisted of a log mat with large brush securely bound on top, the whole being covered with 1 ft. of broken rock. This made a base 40 ft. wide and 3 ft. thick, with the brush tops projecting beyond the sides. At half and high tide it was exposed to the full sweep of the Atlantic. Owing to the lack of an appropriation by Congress, it was left in this condition for two years, during which time the sea washed off nearly all the rock and cut numerous gullies through it. Later, the same thing happened to the north jetty at St. Johns River bar, which was of similar type and on a similar sand bank, but this bank was as much as 2 ft. above low water in one place, and was about 2 miles long. It was intended that the mats, when laid, should be lapped about 6 ft., but, owing to the heavy surf, this was at times impossible, and gaps were left between the two mats. Small mats were used in filling the larger gaps, and rip-rap was deposited in the smaller ones. In many places, especially in the swash channel, these gaps eventually were gullied out to a depth of 10 or 12 ft. At those places the stone was washed out or sunk by the currents cutting away the sand, and mats had to be sunk in them. Judging from these experiences, the writer is strongly of the opinion that, if rip-rap had simply been thrown down, and no mats had been used, the foundation would have been destroyed.

Mr. Ripley's idea of building the foundation in heavy seas (by first throwing down small rip-rap on the sand bottom, to be followed later by larger stone) might be practicable in some small lake, but those who have witnessed the tremendous power of the waves on outer bars on the Atlantic and Pacific Coasts would be loath to try it. As it is known that rocks weighing scores of tons are taken up by the waves from low water and carried nearly to high water, up a steeply inclined beach, it would seem preposterous to expect a wall of small, loose rip-rap, 2 or 3 ft. high, to stand. It would be preferable to lay the larger stones first.

The north St. Johns jetty was afterward built up at the outer end to about half tide, with the result that an excessive scour developed in the swash channel and at the beach, which was eroded for several hundred yards and had to be at once protected by mats and rip-rap at large expense. This would not have been necessary had the jetty been built out from the shore, because, in that case it would have been a groyne, and it is, and always has been, the practice every-

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where to build groynes from the shore out; otherwise they would not hold the sand and build up the beach.

The south St. Johns jetty was built from the shore out, the foundation course being kept well in advance. The re-entrant angle of the beach has been filled solid for a mile out from shore, where the water was formerly 40 ft. deep.

Many years ago, on the Fraser River, in British Columbia, attempts were made to improve the navigation near the mouth by building detached sections of jetty, or training walls, in several places. When the writer examined the river, a few years ago, and made plans for its permanent improvement (now being carried out), these constructions, with a single exception, had disappeared, having been undermined and destroyed because they were disconnected and unsupported, like a small detachment of an army in an enemy's country.

The writer is strongly opposed to such methods, and would not consider the small rip-rap foundation, described by Mr. Ripley, a safe protection at any river mouth on the Atlantic or Pacific, however well it might answer in the Gulf of Mexico or some of the lakes. This, however, is entirely a matter of judgment and experience, for what might be good for one locality might be fatal for another. In sheltered places, where the tidal range is slight, there would not be much danger in commencing the construction of the jetty at the outer end, provided a solid and adequate foundation had been laid first and precaution had been taken to protect the shore end from erosion; and provided, also, that anything was to be gained by it, but the writer's experience shows that there is not.

The value of the isolated section of jetty as a protection to the vessels while working, as claimed by Mr. Ripley, is largely overestimated, as a little reflection will show. According to this plan, the jetty, at high water, will be only 26 or 27 ft. wide at the water surface, and 3 ft. high. If there is any troublesome surf, it will roll over this slight obstruction. At low water the protection would be a trifle better, but, as the scows and tugs depositing stone must be near the axis of the work, it is only when the wind and waves coincide with this axis, in one direction, that it will afford any lee. In other words, the wind blowing across the jetty line would not make any lee for the vessels, though they might work on the other jetty if the wind was right; but this protection would also be afforded if the jetties were being built from the shore outward. When the wind is blowing from the unbuilt part toward the part already built, the jetty will afford no protection. On the contrary, it will be a rocky lee shore, very dangerous to approach. There might not be one day in a month when the wind would blow in the right direction to afford any practical protection.

Mr. Ripley's theory, that, by commencing construction at the outer

Mr. Le Baron. end and working shoreward, the advance of the bar will be prevented entirely and the jetty channel will be deepened more rapidly, is untenable; at least, the writer's experience indicates that it would be very dangerous procedure.

For improvement purposes, all rivers are classed as sediment- or non-sediment-bearing streams. The Mississippi and the Fraser may be taken as types of the former, and the St. Johns, of Florida, as of the latter class. In some places there is a strong littoral current in the ocean in front of a river mouth; in others there is no trace of such a current, or it obtains for only a part of the year. In some places the prevailing winds blow dead on shore; in others they are along shore. In the latter case they produce a wave race along the beach which is fully as effective as the regular currents. All these forces must be taken into account in the study of bar advance.

Now, the author supposes that he has commenced the construction of two parallel jetties over the bar, and states that the material between these jetties will commence eroding and working backward, will produce a deep channel, and "the material thus eroded from the channelway will be carried beyond the outer ends of the jetties, where it will be either swept to one side by the littoral current or deposited in the deep water farther out." He offers no confirmation of this statement, and makes no reference to examples of works constructed in which these movements occurred. On the contrary, the probabilities are that each section of isolated jetty would form the nucleus of a sand island, because no power would be brought into action to direct the river channel between such sections, and it would be just as apt to break out a new channel in an entirely different location.

Channels through river bars are constantly changing. In all bars studied by the writer the channel swings in regular cycles, of from 25 to 100 years' duration, from one side of the entrance to the other, working gradually, for example, down to the south side of the entrance close to the shore, then suddenly breaking out away up on the north side, and slowly working back. The jetty may be planned to run out through the center of the entrance, or to follow the existing channel, which at the time may be close to the south shore. The channel may, and most likely will, jump to a different part of the entrance after some hard storm, and the isolated sections of unfinished jetty will be left standing alone in the water or sand. The foundation line of rip-rap would have no effect in preventing or modifying this change, for the writer has seen the old channel abandoned and a new one formed across the foundation course of a jetty which he built at Fernandina, Fla., and it deepened so much that it was used by large vessels.

Suppose the jetty channel had been laid out to follow as nearly as possible the existing ship channel, that construction had been com-

Mr. Le Baron.

menced at the outer end, and that the channel had shifted as it did at Fernandina. The whole length of the old channel which the jetties had been planned to follow would then fill up, because there would be no jetty to direct the currents, and therefore it would be necessary to build in from sea to shore over a continuous sand bank, covered with breakers at high tide, and impossible for tugs and lighters at low tide without the advantage of a deep channel from which to work; in fact, it would probably become necessary to clear the jetty channel by dredging.

Again, suppose the jetties to have been planned to run straight out to sea over the sand banks—as is often desirable—regardless of the existing deep channel. In this case the same objections would apply, but in greater degree. The writer would like Mr. Ripley to state how it would be practical or even possible to proceed on his plan on a rough and stormy coast.

Under the foregoing conditions, suppose that the entrance, measured on the outer edge of the bar, is 2 miles wide; that each jetty is to be 3 miles long—an average condition—to reach deep water; that work has been commenced at the outer ends, and $\frac{1}{2}$ mile has been completed. This will represent only about 16% of the total length, and will be a very small spot in the waste of waters. It is hardly conceivable that, under such conditions, any scour would take place in the channel until the jetties had been completed to or near the shore. In the meantime, cross-currents will be set up in all directions, the swash channel will surely be deepened, and there will be grave danger of cutting away the beaches, if they are of sand, and low, as is generally the case. Thus the additional dredging and revetting will be almost certain to entail heavy expense.

The filling of the jetty channel can be most certainly predicted, as many bars are formed by the transportation of sand detritus along the coast by littoral currents or surf-race, instead of by river deposition alone. By microscopical examination, Count Pourtales, of the United States Coast and Geodetic Survey, found that the material forming the outer shoals and bars at Boston Harbor was composed of detritus washed and transported from near-by headlands. In all non-sediment-bearing streams this is necessarily the case. In discussing the question of bar advance, one must discriminate sharply between sediment- and non-sediment-bearing streams. In places where there is no littoral current, or a pronounced and continuous surf-race, bar advance will always follow, sooner or later, the construction of jetties at the mouths of sediment-bearing streams, unless the bottom drops rapidly, as it does at the Fraser River.

Mr. Ripley makes no distinction between these radically different types of streams, and appears to ignore the work of the surf-race and littoral currents in their endeavor to perfect the littoral cordon; ap-

Mr. Le Baron. parently, he considers the river bars as formed entirely of river sediment and material washed up from the floor of the ocean in front of the mouth, for he speaks of the position of the bar being determined by the equilibrium of the ocean and river forces, one force (the river) tending to push it seaward, while the other tends to push it back. In reality, these two forces are disturbed by the littoral current (if any) or the surf-race eroding the adjoining beaches and bringing along shore material to assist in filling the river mouth and building out the bar. It is by all these forces that bars are formed, some predominating in one place, and others in another.

The author's claim that, if jetty construction is commenced at the sea end, the bar advance will be minimized, or wholly prevented, appears to the writer to be "not proven," not to say fallacious.

In this discussion, the writer has shown the possibility, or, more correctly, the extreme probability, of the jetty channel being filled with sand in some places, if Mr. Ripley's methods are carried out, and therefore that the quantity of material to be moved will be greatly increased. Mr. Ripley says:

"The material thus eroded from the channelway will be carried beyond the outer ends of the jetties, where it will be either swept to one side by the littoral current or deposited in the deep water farther out."

This is what takes place with any system of parallel or converging jetties, properly spaced, and with sufficient current in the jetty channel to move the sand. In no event will there be any erosion unless the outgoing current has sufficient velocity to move the material composing the bottom. In order to produce this velocity, there must be a head of water.

In the plan advocated by Mr. Ripley, all head is eliminated, because the jetties are at the foot of the slope, in the dead level of the "open sea," and all the slope is shoreward. With jetties built out from shore, on the other hand, advantage is taken of the natural slope of the outlet, which is further augmented by the convergence necessarily given to the jetties by their shore connections; this has the effect of piling up the water in the upper end of the contraction, and materially increasing the head, and therefore the scouring power. In the author's plan this is all lost until the whole system is completed, so that no advantage can be taken of a deep channel for working.

If there is any littoral current, the eroded material will be swept aside just the same, no matter whether the work is begun on the bar or at the shore. The important thing is to induce sufficient erosion to move the material out of the channelway. If there is no littoral current, this material will be deposited, during construction, on the ocean floor in front of the jetties, no matter which method is adopted;

Mr. Le Baron.

and, if the ocean floor is elevated, and the water shallow, the bar will necessarily be advanced in either case.

The claim that this result will not follow if the author's method is adopted, is totally disproved by the writer's experience. The north jetty at St. Johns River bar was laid out by him, and partly built under his superintendence. The foundation course was first built out for about 2 miles. Then the outer end of the jetty was raised to about half tide, where it remained for several years; it was then raised to mean high water, and part of the outer end of the south jetty was also raised above high water before the inner end. According to Mr. Ripley's theory, it would be expected that the bar advance would be retarded, but, as a matter of fact, the 30-ft. curve has moved out nearly a mile in front of the jetties.

A hearting of small stone and the protection of the slopes and top with large blocks have always been good practice. A hearting of oyster shells has been used, and, when plentiful and convenient, does very well. The writer quite agrees with Mr. Ripley in reference to the kind of rock which is most desirable; he also believes in wave washing as a factor in consolidating the stone; and in the importance of making the jetty as nearly water-tight as possible.

Mr. Le Conte.

L. J. Le Conte, M. Am. Soc. C. E. (by letter).—The writer's experience has not been in accord with the author's general conclusions. Jetties are usually built in pairs, converging slightly from the gorge toward the original crest of the bar. At this point convergence ceases, and they run practically parallel, and about 1 000 ft. apart, heading for the shortest distance to deep water—say, 40 ft. at low water. This is given as 40 ft. because that is the depth now demanded in all the main seaports.

For reasons given later, the writer prefers the old-fashioned way of building out from the shore first, until the crest of the bar is slightly overlapped. He would then spread out an apron of rip-rap some 600 ft. beyond the bare end of each jetty, and stop. A long series of observations would follow, in order to ascertain the facts about local conditions. The first thing is to dredge a jetty channel from the gorge out to a depth of, say, 40 ft., and then watch the annual amount of shoaling, which is the only true measure of the necessity of extending the jetties into deep water. This is a most serious consideration, involving great expense. It is always followed by doubtful results, and, therefore, should not be undertaken before making the most painstaking investigations. If the annual amount of shoaling in the channel outside of the bar is not very great, then this extension is entirely uncalled for, simply because annual dredging is now being done so cheaply and expeditiously that it does not pay to extend jetties.

Mr. Le Conte. A little investigation will clear up this conclusion. Theoretically, of course, both jetties should be extended out to a depth of 40 ft. in order to make and maintain a 40-ft. navigable channel without dredging. A conservative estimate of the distance from the crest of the old bar out to such a depth is 3 000 ft., more or less. The cost of extending both jetties from the bar for this distance will probably be $3 000 000, the annual interest on which, at 4%, would be $120 000, which would easily pay the running expenses of a first-class bar dredge for the entire year. Such a dredge would be able to work half the time and would take out fully 600 000 cu. yd. per annum, which is greatly in excess of any shoaling that is likely to take place in 3 000 ft. of channel. In all probability, the annual shoaling would not exceed 100 000 cu. yd. This could easily be removed from the channel for $15 000, which would be the annual charges instead of $120 000, the interest account.

Furthermore, the foregoing assumption is not strictly true, that is, that the extension of both jetties to a 40-ft. depth would probably do away with dredging entirely. Experience almost everywhere seems to show that, in nine cases out of ten, in course of time a new bar will form across the 40-ft. entrance; so that probably some dredging will have to be done in addition to the heavy interest charges of $120 000 per year.

A practical study of the existing facts in jetty construction will invariably end in omitting any proposed extension into deep water, because, from a purely financial point of view, there is absolutely nothing in it.

The writer, however, agrees with the author about many things, namely, framing the specifications so as to make use of the entire output of the quarry, because this has to be paid for in any case, and why not make use of it; also, his remarks about the hearting wall and big stone facing on the slopes, and maintaining the apron out in advance of progress work. The superstructure from the low-water plane up to the crest of the jetty can now be built, with much better satisfaction, of monolithic concrete reinforced at and near the outer ends.

Mr. Ripley. HENRY C. RIPLEY, M. AM. SOC. C. E. (by letter).—The writer is much pleased at the generally favorable consideration which his paper has received. It was hardly to be expected that no differences of opinion would be drawn from widely different experiences, and it is hoped that any adverse criticisms which have developed in the discussion will contribute in the end to a fuller elucidation of the subject.

The writer agrees with Col. Abbot that some large stones may be used in the core of the work. It is often necessary to do so in order to preserve the core intact until it can be protected by the side- and crest-blocks. But he does not agree that "it is well if possible to delay the final covering with the largest blocks until there has been time enough for ocean waves to develop weak spots." Mr. Ripley.

A properly constructed jetty should have no weak spots. Its only weakness consists in allowing it to be exposed unduly to ocean waves in an unfinished condition.

Mr. Taylor has explained some of the difficulties encountered in the construction of works of this character, where there is considerable sea to contend with, such as is found on the Atlantic and Pacific Coasts of the United States, and requests information as to the conditions and degree of exposure under which it would be permissible to use such steep side slopes as are given in the paper.

It seems that Mr. Taylor does not distinguish clearly the difference between a rubble mound and a scientifically constructed work, such as the writer has attempted to describe. The former will possess stability directly in proportion to the weight of the pieces of stone used and to the gentleness of the side slopes, while the latter will depend for its stability largely on the way in which the side- and cap-blocks are placed. The jetty at Coos Bay, quoted from Mr. Tower's paper, was evidently nothing but a rubble mound. Had this jetty been constructed in accordance with the rules laid down in the paper, with side- and cap-blocks weighing from 15 to 30 tons each, and with an apron extending well beyond the outer end so as to prevent undermining, it would have stood up against the heaviest waves of the Pacific Ocean.

Some years ago the writer designed a breakwater for the protection of Manzanillo Harbor, Colima, Mexico, for the Mexican Central Railway Company. Before it was constructed, however, the work passed out of the control of that Company, and the plan was somewhat modified. As completed for a length of 1 446 ft., the sea slope was covered with concrete blocks weighing 30 tons below and granite blocks of somewhat greater weight near the top. These granite blocks were as large as could be transported on cars from the quarry. The inside slope was covered with granite and concrete blocks of from 5 to 15 tons, and the whole was capped by a concrete monolith 27 ft. wide and 13 ft. thick.

Fig. 1 is a cross-profile of this breakwater, the information for which was kindly furnished by Edgar K. Smoot, M. Am. Soc. C. E., who constructed the work by contract for the Mexican Government. This cross-profile was taken near the outer end of the breakwater, and shows a base width of 315 ft., a height of 87 ft., and a side slope on the sea side a little steeper than 2 horizontal to 1 vertical.

This work is said to be "absolutely impregnable to the action of the ocean waves." It was commenced in 1900, and has now been finished for several years. If this work, exposed as it is to all the fury of

Mr. Ripley. Pacific Ocean storms, will endure, why should any similar work be designed with greater side slopes than 2 horizontal to 1 vertical?

The curved breakwater at Aransas Pass, Texas, built by the writer, as contractor, has a side slope on the sea side generally less than 2 horizontal to 1 vertical. The cap- and side-blocks are of granite, the former varying from 10 to 15 tons and the latter from 5 to 10 tons. The top width, for a height of 20 ft., is about 15 ft. This work was constructed in 1903-04, and has successfully resisted the waves without injury. Such a structure should resist the action of such storms as the Galveston cyclone of 1900, in which the wind reached a velocity exceeding 100 miles per hour and the waves disturbed the bottom at depths of from 30 to 40 ft.

The side-blocks may be put in place below water with a derrick, where the depth is not great, and, where the water is deep, by allowing them to roll down the slope; and this may be done to advantage in any case below low water, as the momentum acquired in rolling down the slope tends to embed each block firmly in the foundation, or jam it solidly against those below.

CROSS-PROFILE, MANZANILLO BREAKWATER, MEXICO

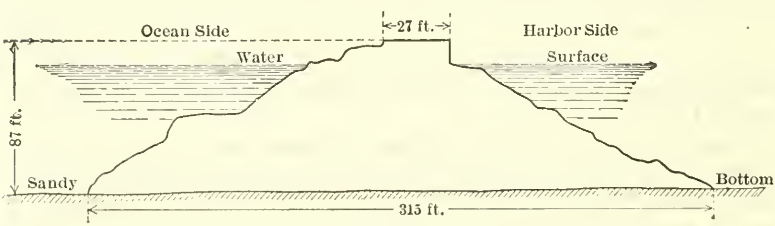

Fig. 1.

For the outer ends of jetties which are much exposed to current and wave action, an apron must be provided, the width of which will be governed by the probable depth of scour to take place, and somewhat heavier blocks should be used than are required for the sides. With these precautions, the end will remain as stable as the rest of the jetty.

The proper top width to be given to a jetty will depend on the degree of exposure and method of construction. Generally, however, if the top width is equal to the height of the work above the foundation, from 6 to 24 ft. will suffice, 6 ft. being the minimum width for lesser heights and 24 ft. the maximum width for greater heights. In most cases, however, the top width will be governed by the dimensions of the core, and the greatest care must be exercised in construction to prevent the spreading of the core beyond the limits which will permit the top widths here indicated. The top width as here understood is that width determined by the intersection of the planes of the sides with the horizontal plane of the top. The fact that the corners may be rounded off, as it is desirable they should be, does not in any way affect the theoretical top width. Mr. Ripley.

Mr. Tower objects to side slopes of 1 : 1¼ on the sea side and 1 : 1 on the channel side, as being too steep for general application.

He does not realize, perhaps, that these slopes are applicable only to the apron, which is practically free from wave action. On the sea side, the apron, if undermined along the edge by the overfall or current action during construction, will settle down into a trench below the general level of the bottom, and will thus be beyond the reach of the waves. The channel side is generally out of the reach of wave action. Where these conditions do not exist, a gentler side slope may have to be provided for, or it may be necessary to use larger stones. In every case it is necessary to give due consideration to local conditions.

Mr. Tower disagrees as to the order of construction for the superstructure which the writer recommends, that is, in commencing at the outer end and advancing toward the shore, and opposes some theoretical objections to such a method. The writer has been connected with two important works in which this order of construction has been followed, and in neither case were there any of the unfavorable results which Mr. Tower predicts.

The Galveston south jetty, 4¼ miles long, constructed of brush mattresses and stones, was completed by building from the outer end toward shore, after the foundation (about 2 ft. thick and 120 ft. wide) had been laid.

At Aransas Pass, Texas, the curved jetty had the same order of construction, except that for a short piece the specifications were not complied with. The writer, under contract with the Government, built about 2 000 ft. of this work, commencing at the outer end. In placing the crest- and side-blocks, it was found that about 250% greater progress could be made in working toward shore than in the opposite direction, on account of the greater protection afforded in the lee of the finished work. By this order of construction, bar advance at that point was eliminated.

The outer end of this jetty was located in 15 ft. of water, and its position has never been changed; and yet, to-day, the 20-ft. curve is inside of the outer end of the jetty, and at the end there is a depth of more than 30 ft. A little shallower depth is found outside of this, but a clear depth of 24.8 ft. can be carried from the end of the jetty to sea. Mr. Tower, however, says there are many places where this order of construction is entirely impracticable. If this be true, that settles it; but the writer has had experience on the Atlantic, Pacific, and Gulf Coasts, and it is his judgment that, where it is possible to work from a trestle at all, if the trestle be made high enough above the water, it can be made strong enough to resist all ordinary

Mr. Ripley. storms, if not the periodic cyclones which visit the Gulf of Mexico only once in several years.

Mr. Le Baron finds much to criticize in this paper. He admits, however, that it gives in small compass some recognized facts and practical information in reference to jetty building in the open sea, but does not seem to present any facts not previously well known and tried by harbor engineers. He finds the writer's definition of a jetty very circumscribed, and says there are more than a dozen types of construction used in building jetties in the open sea, some being adaptable to one place and some to another. He himself has been connected with some works in which the types have included:

"rip-rap rock, log-mat and rock, brush mattress and rock, rock and mud-filled cribs, plank and pile jetty, and log and pile jetty; and, on some of the large works, three of these types have been combined in one jetty."

He advocates the use of log-mats for foundations for purposes of economy, while admitting that stone alone will not sink more than 1 ft. into the bottom. He finds the proportions of jetty described by the writer too small for any exposed place. For outside work on the Atlantic and Pacific Coasts, he has always used a top width of 40 ft. and side slopes of 2 : 1, and believes that in many locations 4 : 1 is better for the outside. He also finds that log and brush mattress in the apron is better than stone.

The writer did not undertake to describe more than one kind of construction, on one character of bottom, and having one degree of exposure, namely, the open sea. He did this without prejudice to any other kind of construction which may be suitable for bottoms of other characters or lesser degrees of exposure.

Inasmuch, however, as the question has been raised as to the desirability of log-mats and brush mattresses in work of this character, it may be interesting to note some of the results of actual practice in the use of log and brush mattress on the Gulf and Atlantic Coasts of the United States.

Between 1880 and 1885 about $2 000 000 were expended at five places on the Texas Coast in brush-mattress work weighted with stone, bricks, and concrete blocks. At Galveston, where about one-half of this total amount had been expended, the structure had settled more than 50% one year after work was stopped. At Brazos River, Pass Cavallo, Aransas Pass, and Brazos Santiago, where the remainder of the money was expended, the work had completely settled beneath the sand. At Pass Cavallo the settlement was so great that the officer in charge recommended, in case further work was to be done, that a new site be chosen for the jetty, as it would cost less to start in a new place than to build up from the old work, the top of which was below the level of the original bottom.

The Galveston jetty was afterward rebuilt with stone alone, in the manner described in the paper, and became a permanent work. The Galveston north jetty, 5 miles long, was built entirely of stone, scour in advance being prevented successfully by keeping the foundation about 1 100 ft. ahead of the completed work. Mr. Ripley.

At about the time the brush-mattress work was going on in Texas, or a little earlier, there was inaugurated on the Atlantic Coast, principally at Charleston, S. C., at the mouth of the St. Johns River, Florida, and at Cumberland Sound, Georgia and Florida, a system of jetty construction consisting of log-mats weighted with stone. This was modified at some places by brush mattresses, and the system was continued for more than 20 years. The Charleston jetties were finally completed with stone. The Cumberland Sound jetties were built up and repaired repeatedly, until finally the south jetty was abandoned altogether and the north jetty was successfully built of stone alone, the brush works serving only as an uncertain foundation.

It seems incredible that an engineer of Mr. Le Baron's experience would be willing to defend, much less advocate, a system of construction involving log- and brush-mats, after the expensive and disastrous experiences on the Atlantic and Gulf Coasts. Mr. Le Baron himself, in a discussion of a previous paper before this Society, has told how a section of the St. Johns River jetty was undermined and settled bodily some 20 ft. This jetty had a log-mat foundation, and yet he advocates at this late day a log-mat and brush foundation and apron in preference to those of stone alone. It is doubtful if a single member of the Corps of Engineers of the Army who has had experience with log- and brush-mats on the Atlantic and Gulf Coasts would be willing to repeat the experiments of 20 or 30 years ago.

Mr. Le Baron's objection to the proportions of the jetty described by the writer, preferring a top width of 40 ft. and a side slope on the sea side of 4 : 1, brings out in a forcible manner the economy of a scientifically constructed work such as the writer has described.

Much stress is laid on the heavy waves of the Atlantic and Pacific Oceans, and the difficulties in carrying out work of this character, by Mr. Le Baron and some others in this discussion. The writer has had experience on both these coasts, and, while he is aware of the force of these waves, he does not hesitate to express the belief that on either coast jetty construction can be carried on in accordance with the rule laid down in this paper; but, the larger the waves, the larger must be the stone used, the stronger must be the trestle, if done from a trestle, and, if done with the floating equipment, larger barges will be required.

As an argument against the use of small rip-rap in the foundation course, Mr. Le Baron says:

Mr. Ripley. "As it is known that rocks weighing scores of tons are taken up by the waves from low water and carried nearly to high water, up a steeply inclined beach, it would seem preposterous to expect a wall of small loose rip-rap, 2 or 3 ft. high, to stand."

The writer's experience has taught him that a solitary stone on a sandy beach subject to wave action would be undermined and settled down beneath the sand without any appreciable horizontal movement, much less a movement up a steeply inclined beach; hence he must regard this phenomenon as outside the scope of natural laws and to be dismissed as miraculous. The engineer can only deal with natural forces under laws which can be studied and determined. It may have been some such phenomenon as this that General Gillmore had in mind, when he designed the Charleston jetties, in providing chains securely anchored in the bottom course of log-mats with which he proposed to hold the crest-blocks in place. As the work progressed, however, the chains were abandoned, and when the work came to be capped it was found unnecessary to use any anchorages whatever to hold them in place.

Mr. Le Baron says:

"The author dismisses the question as to the effects of such storms in a näive manner, as 'problematical.' In proportioning the width of the apron, he estimates that the damage from overfall is caused by the waves only."

As neither of these statements appears in the paper, it is unnecessary to reply, further than to correct the misapprehension.

Mr. Le Baron further says:

"Mr. Ripley's theory, that, by commencing construction at the outer end and working shoreward, the advance of the bar will be prevented entirely and the jetty channel will be deepened more rapidly, is untenable; at least, the writer's experience indicates that it would be very dangerous procedure."

The writer wishes to state emphatically that there is not any "theory" in his paper. It is a simple narration of the method used by him in actual work, every feature of which has been tested by time; and, as far as he knows, it is the last word in this class of construction. Elsewhere in this discussion he has mentioned two important works, namely, Galveston and Aransas Pass, where the superstructure was commenced at the outer end and progressed shoreward, an order of construction which Mr. Le Baron says is without precedent. At Aransas Pass, during the writer's contract, the capping was commenced for convenience at a point 250 ft. from the outer end, and the work was extended outward until this 250 ft. was finished, whenever the weather would permit working in that direction, otherwise, the capping was extended shoreward. Now the record shows that it

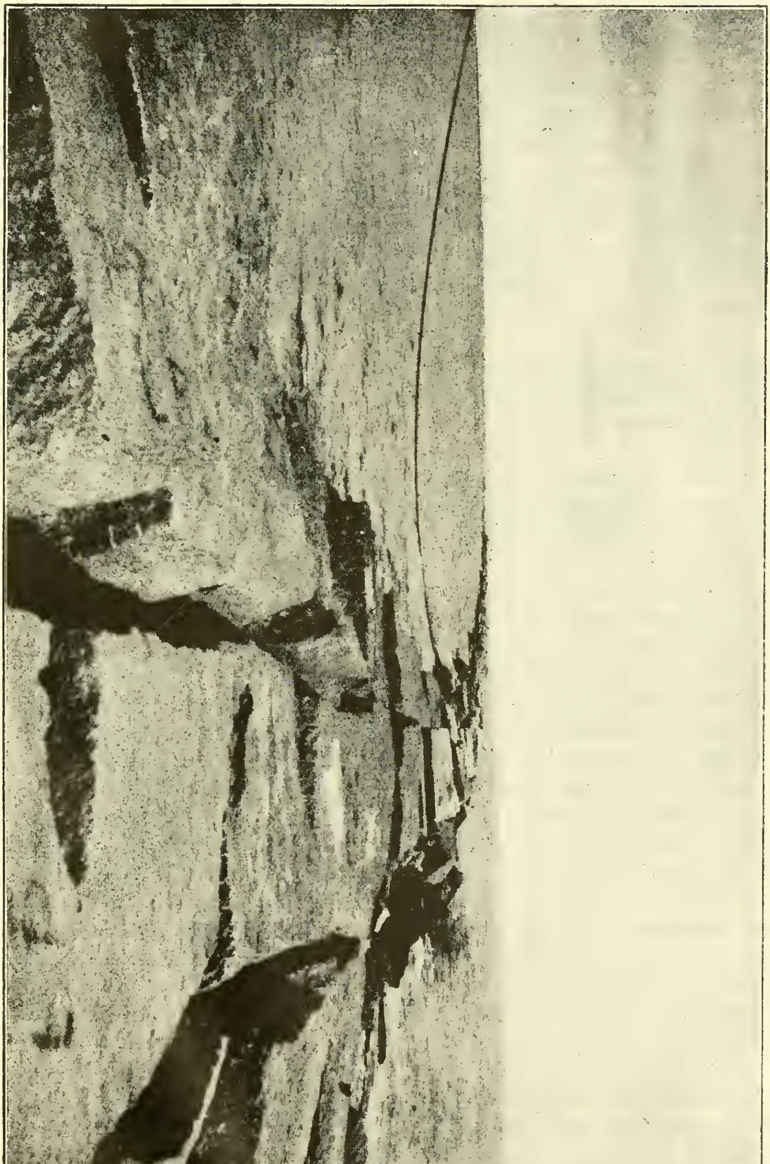

FIG. 2.—ARANSAS PASS CURVED JETTY. LOOKING SHOREWARD, 1904.

took some 2 months to complete this outgoing work, while in the same time 875 ft. of ingoing work were completed. That is, it was possible to complete three and one-half times as much work by working toward shore as when working outward. This is fact, not theory. Mr. Ripley.

Now, as to the advance of the bar being prevented by this order of construction, it is recalled that at Aransas Pass, where this was done, the jetty was commenced in 15 ft. depth, outside the crest of the bar. The latest official survey, that of June, 1912, shows the 20-ft. curve of depth 600 ft. shoreward from the outer end of the jetty on the side away from the channel, while the depth in the channel at that point is more than 36 ft.; and going directly seaward from the outer end of the jetty, the least depth is more than 23 ft. Here, again, is a fact, and not a theory, as an answer to Mr. Le Baron's theoretical objections.

Mr. Le Baron prefers a monolithic concrete construction from low water up, with reinforced concrete at and near the outer ends. The writer does not think this method nearly as good as separate blocks, for the following reasons:

First.—If constructed before settlement has ceased, the monolith will either break in two or span low places, and so fail to give the required protection to the mass below; or, where settlement is great, it may break up and tumble down the sides of the wall into the deep water at the base. Some years ago the writer had occasion to examine a breakwater at Vera Cruz, Mexico, which had been wrecked by a severe storm. This work was about 1 mile in length and extended from the shore to an island. It was constructed of 30-ton concrete blocks placed immediately on the sand in an irregular manner and brought up to the water surface. On top of this was a monolithic concrete construction about 20 ft. wide and 15 ft. high. When the storm came, the sand at the base washed away and the whole mass settled irregularly. The monolithic superstructure was broken into irregular pieces and tumbled down the slopes on either side. This, of course, is an extreme case, but it illustrates what must occur in a lesser degree in any work of that character when settlement takes place.

Second.—If the superstructure be delayed until settlement has ceased, the work must be left incomplete for a very considerable length of time, and is likely to suffer serious damage from storms. Such a method is only applicable to rubble mounds with gentle side slopes which greatly increase the bulk, and therefore the cost, of the work beyond that required for a finished structure.

Third.—It is more expensive to construct a monolith on the jetty itself, subject as it is to the interference of waves and part of the time under water, than to construct separate blocks in the yard and afterward place them in the work; and the separate blocks are otherwise preferable.

Mr. Ripley.

Fourth.—Particularly objectionable is the reinforcement of the monolith at the outer end of the jetty, because it will delay the breaking of the monolith where the settlement occurs at the end, and so deprive the work of the support which the crest-blocks are designed to give, and this, too, at the most vulnerable part of the jetty. It may be likened to a stiff brush- or log-mat which permits the sand to wash out from under its edge until such a point is reached that the weight above breaks or bends the mattress down into the trench, or until, as in the case of a log-mat, the undermining passes the center and one side tips down into the trench while the other side tilts up, which, in turn, is undermined, and bodily settlement is the result.

With the reinforced monolithic superstructure, also, its very stiffness prevents the end from following the settling mass below, and so permits the waves to wash out the smaller stones beneath until finally the unsupported length breaks off and comes down with a crash on the submerged mass below.

The writer is aware of some important works with monolithic capping which have not suffered materially on that account; but the principle is wrong. The properly constructed jetty must be perfectly plastic in all its parts, from the foundation to the crest-blocks, so as to accommodate itself to any tendency toward undermining or settlement. Each crest- and side-block must support and be supported by those adjacent to it, and must be placed so that the waves will glide over rather than meet resistance from unnecessarily exposed surfaces. Fig. 2 is from a photograph of the Aransas Pass curved jetty, and shows, better than words can describe, the way to place the crest- and side-blocks so as to fulfill the requirements above specified.

The writer does not agree with Mr. Le Baron as to the desirability or necessity of securing and maintaining a jetty channel by dredging; but this is a matter wholly outside of the subject of the paper, and, as it is feared that the reader's patience will have been exhausted, the writer foregoes the temptation to branch out into this interesting field. However, he hopes in the near future to present to the Society a paper on the subject of the proper location of jetties on an ocean bar in order to secure and maintain a navigable channel across it without resort to dredging.

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TRANSACTIONS

Paper No. 1238

ENGINEERING EDUCATION IN ITS RELATION TO TRAINING FOR ENGINEERING WORK.*

By Ernest McCullough, M. Am. Soc. C. E.

With Discussion by Messrs. N. B. Garver, George B. Pillsbury, F. H. Constant, Arthur B. Green, Alexis Saurbrey, J. X. Cohen, George F. Swain, William J. Boucher, Almon H. Fuller, Walter Hinds Allen, C. H. Stengel, Charles Warren Hunt, Arthur H. Blanchard, Philip W. Henry, John C. L. Rogge, Charles H. Higgins, Charles B. Buerger, and Ernest McCullough.

It is conceded that the true engineer is a man of action, resourceful, ingenious, and possessing considerable executive ability; the sort of man who should succeed, whatever line of work he might have been thrown into at the beginning of his career. Computing quantities, calculating strains and stresses, making minor experiments, etc., as a rule, is not paid for at any higher rate than ordinary clerical work, while advancement is well nigh impossible for the majority of men who confine themselves to the minor details of engineering design and construction.

That the foregoing facts are not fully recognized in engineering schools is apparent to the majority of engineers in active practice, hence the amount of criticism leveled at them and the men who are conducting them, much of it unjust, and much of it merited. The writer does not wish to be classed with those who are wholesale in their criticism of engineering schools, but he does believe that the time has now come when a number of changes can be made, to the advantage of the schools and of the graduates of the future. The country now being well supplied with men trained to do engineering

* Presented at the meeting of September 4th, 1912.

work, the schools can well afford to stop advertising for students and concentrate their best efforts on a selected number who show promise of ability and, to use a common expression, "gumption."

The practicing engineer is always reminded by teachers that it is impossible for him to obtain the viewpoint of the teacher, there being two distinct divisions in the Profession, engineer and engineering teacher. This may be true, and the rebuke of the teacher generally silences, though it may not convince, the engineer, until he examines carefully the work of the teacher, whose sole function in life is to prepare assistants for the engineer, and train those who in the future will be engineers. An examination of the curricula of seventy-five schools of engineering shows that some startling differences of opinion exist among teachers themselves. The writer has made such an examination, and his conclusions confirm those of Mr. Harwood Frost, made a few years ago.* These schools are not considered inferior, many of them being of high standing.

Briefly, taking schools claiming to give complete courses in civil, mechanical, electrical, and mining engineering, and considering the minimum time given at each school in each subject, and then considering the maximum number of hours given in a number of schools, it is found easily possible to create a six-year course of study in which the students will be given an education in all the branches of engineering above mentioned. This shows, not only the overlapping of studies in each branch, but also illustrates in a remarkable manner the differences of opinion existing among teachers as to the relative importance of each subject taught. In some schools at present advertising five- and six-year courses, a careful investigation of their catalogues shows that there is really no more time given than in other schools having four-year courses, the catalogue hours in the former case being 15 and 17, and in the latter 19 and 21. In the schools having the five- and six-year courses much stress is laid on the number of "culture" studies given as compared with more purely technical schools, yet the total number of catalogue hours in each course is very nearly the same, the longer courses merely giving the instruction in smaller doses. If there is any greater amount of "culture" studies, then the students are defrauded of some part of

* Published in *Engineering News*.

their technical work; or, in the other schools, undue importance must be placed on some technical studies, slighted in the schools combining culture and technical training.

The writer believes the engineering schools of the future in the United States will probably call for a minimum of six years' work. The algebra, trigonometry, analytical geometry, drafting, shop work, elementary surveying, physics, and chemistry, as now taught, will be given in technical high schools and other preparatory schools, which means an added two years of preparation. These subjects being required for entrance, the engineering schools can maintain their four-year courses and make good use of the time now spent on the foregoing elementary subjects. Such schools will develop into research institutions where the best of training can be given. All-year sessions, such as are now in vogue at the Michigan College of Mines and at the University of Chicago, will be the rule. Students may then elect to attend school four sessions of 12 weeks each year, with one week of vacation between terms, and complete their work in three calendar years, or may take three terms per year and complete their work in four years, or, if financial considerations require it, may take even a longer time, entering newly started classes in any term. This will permit a sandwiching of work in the shop, office, field, and school, with great benefit to the students. Arrangements may ultimately be made between the schools and business establishments and corporations, so that the alternating of work and study may be required and provided for. At present all schools cannot be so fortunate as the University of Cincinnati in this respect.

For some of the older students, who do not work well in classes, the plan of Dean Raymond for teaching by seminar methods will be a part of the training of the engineers of the future. It is well known that a great many men, fit to be first-class engineers, need something of this sort to awaken them, and, if teachers of the right kind are provided, they will be saved to the Profession and to themselves; whereas under present academic methods, they are "flunked out," and handicapped for a long time, until they prove themselves in the hard school of experience.

The future will work out its own problems. We are now dealing with the present. The Profession is filled with unfit men, due to the fact that the majority of teachers insist that, since they are running establishments to give engineering instruction, the engineering instruction must be given to all who apply. Unfortunately, the instruction must be given in one way, and in one way only, to all, regardless of all the little things that go to make up the difference between one man and another. The length of the present standard term of study is four years, and there is no standard course, each school being a law unto itself in that respect. The subjects are given in the customary academic style, and the atmosphere in all schools is that of the school and not that of the office. It is the lack of office atmosphere that engineers deplore, although, in the numerous criticisms made of engineering schools, this fact is not brought out in plain words. It is not only the unfit men we should consider, but the vast number of disappointed men in the ranks.

It should not be a difficult matter for teachers to standardize a course of instruction in engineering. There are certain fundamental subjects on which they are all agreed, and it is merely the difference in emphasis placed on the subjects, according to the ideas of men in charge of departments, that constitutes the differences in the schools. There is also the desire of so many professors to create departments in which the head of the department will be able to obtain considerable glory as compared with other men on the faculty. Each specialist magnifies his specialty. Within the past month the writer has had submitted to him a proposed syllabus for a course in testing engineering, for the training of laboratory workers. Let us see how this works out:

A well-known university compels all engineering students to make a choice at the end of the freshman year from the following specialties: Architecture, architectural engineering, architectural decoration, civil engineering (general), electrical engineering (general), mechanical engineering (general), municipal and sanitary engineering, railway civil engineering, railway electrical engineering, railway mechanical engineering. In the last issue of the periodical published by the associated engineering societies of the university appears the following editorial, apparently written by one of the professors:

"The laxity of organization among the alumni after graduation is in part due to the system in vogue among the various colleges. In the Engineering College there are more than half a dozen individual societies, with two sometimes existing in the same department. Little opportunity is given for members of the different departments to meet, and it is no wonder that many an Engineer graduates without knowing all the members of his Engineering Class.

"Little opportunity is given for those who desire to act as a unit while occasions exist. The Engineering Dance is a function that is carried on by the Societies, yet few know how the committee is selected. Many worthy lecturers attend the University, and are greeted by but a few enthusiasts, due to the fact that the lecture is advertised as a specialty of a single Society. Too many in fact, of the Engineering Students relax with the idea that a mere specialty, a particular course, is sufficient for them, only to regret such an act at some later time.

"By looking over the records of our alumni, we are surprised at the large number that engage in work entirely different from that which they took up in college."

The editorial continues with suggestions designed to remedy the criticisms made, but nothing is said about the influence of the university catalogue, with the large and confusing number of specialties from which the bewildered freshman is compelled to select. The writer desires to call attention to the last paragraph of the foregoing quotation, as being the one thing of which the greater number of professors seem oblivious, that a good general course well taught is all that is essential, and that special work should be graduate work, given in one or two terms in the winter, when so many engineers are out of employment and could profitably spend a few months in investigative and laboratory work, after having settled for themselves questions of momentous importance. The function of the college is confused with that of the university. The writer of the foregoing editorial expresses surprise at something so well known to practicing engineers that the majority of them may be pardoned for feeling slightly contemptuous when reading the article.

"Choice and Chance" are mathematically dealt with by professors, and some of us can remember the time when we spent hours in solving problems in these branches of mathematics. If one were to take down the dust-covered algebra and refresh his memory with the problems in Choice and Chance he would wonder far more at the professor who expresses surprise that so many men who made a choice were later compelled to change because of the mutations caused by the greater influence that chance exerts on the life of that nomad, the engineer.

That a school training is of immense advantage to an engineer no one denies, although we have so many first-class engineers who have had no adequate school training that no one insists on the school work as being an absolute necessity. It is not and never will be; but he is a stronger man who adds to his natural ability the power given him by the pursuit of a carefully arranged course of study in the essentials of engineering science at a well-conducted school. This being the case, it should be settled by engineers, and not teachers, just what instruction is best and how it should be imparted. This may lessen considerably the importance of the teachers, but who can say that this will be harmful? Teachers are supposed to train young men to be proper engineering assistants, who may develop, if they have the opportunity, into successful engineers. The engineer should merely give to the teacher his specifications for a good assistant, and the teacher should try to follow the specifications. The engineer factories —the schools—at present insist on the acceptance of their output, produced according to their specifications; while the consumer is ignored, or contemptuously classed as a "kicker." Manufacturers of materials used by engineers have been compelled by competition to furnish products satisfactory to the user, so why should not the schools, that manufacture embryo engineers?

It is not claimed that all, nor even a majority, of the products of the schools is not satisfactory. Very fortunately, an immensely large number of young men attend engineering schools who are endowed by Nature with good brains and considerable common sense and aptitude. The complaint is made that by far too large a proportion of unfit material is run through the factory; that is all.

The writer believes that for the present we should deal with the four-year course, leaving for the far future and the well-endowed university the problem of gradually altering the course of instruction so that, step by step, the ideal engineering school of the future will materialize. Perhaps half a dozen such will serve the country, and this will leave the remainder of the more than 200 technical schools to adopt some such plan as is here proposed.

The high schools are complaining that their courses have to be arranged as though all the pupils were to enter college. This narrows the curriculum, so that the high school cannot be properly called "The People's College." The best definition of education the writer has ever seen is one credited to an old engineer in a factory, who said that in his opinion a man was well educated when he "was on to his job and could make good." The engineering schools, therefore, should demand for entrance requirements merely algebra through quadratics, plane and solid geometry, plane trigonometry, high-school physics, and chemistry, and should permit the student to offer, for the remaining credits necessary to make up the fifteen now demanded, any subjects that happened to have been taught in the particular school he attended. These credits could be: ancient and modern languages, geology, physical geography, English, of course, bookkeeping, shorthand, or, in fact, anything that required a certain number of hours of class work, sufficient to make a major in high school.

In the technical school foreign languages as required subjects should be omitted. The methods of teaching languages in the average college and technical school leave much to be desired. The subjects should not be forced upon students merely as devices to gain credits. The extremely small percentage who may go to foreign countries can readily pick up a working knowledge of any language in a few weeks by means of the phonograph and special schools. The writer knows many who have done this, and the fact cannot be denied. It is not necessary to learn a foreign language for any other than conversational purposes, for everything of value appearing in the foreign papers is quickly translated. The advocate of the study of languages for "culture"—whatever that may mean to any one who is not a bacteriologist—pleads that those who do not study foreign languages miss much that is choice in literature. When all the good English literature has been read it will be time for the soul thirsting for such reading to prepare himself for a feast of reason. The study of languages should be a matter of individual preference or selection.

The average student in a technical school is of no more use to an employer at the end of his second year than a high-school graduate. Financial reasons compel a large number of students to leave, and many do so because they are unable to keep up their work to satisfy the standards set by the faculty. The majority of these men go into offices as draftsmen or to assist in other ways, and some do amount to something later in life. The writer does not intend by stating these facts to endorse the old saying that "the most successful student at college generally fails of success in business life." That saying originated in schools giving the so-called liberal arts courses, in which the courses of study were not supposed to prepare a man for any useful service in life. It was quite easily possible that a bookish man could become so impressed with the immense importance of his purely book work that practical affairs became distasteful. The young fellows who worked merely that they might secure the necessary credits to entitle them to a diploma showed acumen that stood them in good stead in after life. The contrary is the case with technical schools. Invariably, the best student is the most successful engineer, as may be discovered by searching the college records of successful engineers. There are a few, of course, who stand high in scholarship and fail to meet the conditions of the working world, but it will be generally found that these men succeed at school in the purely academic subjects, or in those requiring minds schooled to carefulness in extremely small details. That is, they have the minds of clerks rather than those of managers, therefore the executive work of the engineer is rather badly performed by them.

However this may be, a few men do not waken properly to their responsibilities during the first two or three years in school, but are successful in practical life. A school for the training of men in such very practical subjects as engineering should be organized to take care of all who enter. Each year should see the student a little bit better prepared in some one of the many things he is set at, instead of, as now, making them work for two years almost entirely on preparatory studies which are to fit them to carry on the work of the last two years properly.

Were the writer to arrange a course in engineering, he would aim to give such instruction in drafting during the first year that every student should be able to go into the offices of architects, civil, mechanical, or electrical engineers and earn the pay now earned by the average draftsman. To become such draftsmen does not require any large amount of mathematics, and the amount studied at the preparatory school, and required for college entrance, is amply sufficient. The writer would require the students to work in a drafting room under conditions approximating as nearly as possible actual working conditions. Four hours each day should be spent in this work, of which one hour should be devoted to lettering for the first semester. This should be done for five days each week, the afternoon of the sixth day being spent on the campus learning the duties of rodmen and chainmen, the men in higher classes acting as instrument men and chiefs of parties.

The instruction should be such that the boys would learn as much as possible about standard connections and methods of drawing details in structural work, the steel companies' hand-books giving plenty of examples. They should also take about the same kind of instruction in architectural details and in the standard details in machine shops, etc. This should be merely a course in drafting, and not one in design; only the most elementary principles of projections should be taught. In the summer vacation nearly all should be able to get work in drafting offices at fairly good pay, and if half the boys do not return to school in the fall, because they have become launched in life on work they can do well and have no ambition to do better, the Profession gains most decidedly. The writer recently wanted to secure a draftsman for temporary work and was unable to get the man he required although he interviewed more than forty applicants. Three were graduates, about twenty had been more than one year in an engineering school, and the remainder were graduates of technical high schools. None was a good draftsman and none was ambitious to be more than a draftsman.

In the mornings the mathematical instruction will be that usually given in the first year. Physics and chemistry will not be given in that year, the high school study in those subjects being sufficient for the understanding of many things. Instead of the academic courses in these subjects, the students should study books like Munby's "Physics and Chemistry of Building Materials," and Sorsbie's "Geology for Engineers." These should be studied thoroughly and occupy the whole year. One hour each day should be spent in going through the examples in Sanborn's "Mechanics Problems."

The foregoing will keep the students busy, but the year will be a most interesting one, for they will see the applicability of each subject to their life work, something denied them now in the majority of schools.

It is assumed that half the students will return the second year, for many will obtain in the first year about as much as they ever hoped to gain at school, and, during the vacation, many will have secured remunerative positions. All, however, should be young men who will bring no discredit on the school, as their training will have been of such a practical nature as to invite the respect of their employers. The general complaint against the average graduate is that he cannot solve readily ordinary arithmetical problems, while he may be able to chase the elusive x and y through the mazes of a cubic equation with bewildering facility.

In the second year, physics and chemistry will be taught, and the calculus, as is usual; but the study of mathematics should be pursued in a somewhat better way than is customary in American schools. Either mathematics should be taught in a manner that will provide the student with a useful tool, or the time should be given to some other subject. American schools are remarkably deficient in the character of the mathematical instruction given, but this is so old a complaint that there is no need to press it here. The teachers themselves are wakening to this fact, but the writer firmly believes that the study of a book like Saxelby's "A Course in Practical Mathematics" is amply sufficient for the needs of all engineering students, and can be safely begun in the Freshman year and be completed in about three terms. The use of the word "omit," which so many teachers favor, should be abolished in connection with the text-books on mathematics. A noted American professor sneered at this book of Saxelby's, yet the writer had in his employ two graduates of this professor who were not possessed of a working knowledge of the subject. At the same time, he had in his employ an Englishman who had used Saxelby's book as a text, and had no more trouble in using the calculus than in using ordinary arithmetic. His two companions, the Americans, purchased copies of this book in order to review their work, and were loud in its praise, soon becoming as proficient in the actual use of the calculus as any employer could desire. It must not be inferred that we had any more use for the calculus on the work we were doing than is common, but the foregoing facts were discovered in talking with the men. Graduates require more exercise in the doing of problems, and the drill during the first year in problems in mechanics should be followed in the second year by those which are apt to arise every day in actual work.

During the second year the students should study surveying well, and put in three afternoons every week on the campus practicing. The first-year students should be well drilled in the work performed by rodmen and chainmen, who need no knowledge of other surveying operations, and they should do this work for the men in the higher classes. During the first semester the use of the level and compass should be taught, with plenty of practice, and in the second semester the use of the transit. The other three afternoons should be spent in map drafting and the making of profiles, estimates for grading, estimates of quantities in structures, etc. In the first semester one hour per week should be devoted to a study of various methods of keeping time and the reading of books on management engineering and cost-keeping systems. This work should not be exhaustive, but should be such as to acquaint the student thoroughly with methods for finding information on such subjects when wanted, and familiarize him with the appearance of the common forms used in business. One hour each week should be spent on the study of engineering English, with as many examples given of poor specification writing as it may be possible to obtain. A certain amount of reading of masterpieces of English literature should be required, say one each month, and just enough of a quiz should be given on each book to be sure it was read. The well-read man is generally able to pose as a "cultured" man, and, if a liking for good reading can be instilled into the students during their college days, considerable will have been done.

In the third year structural design will be taught, and also strength of materials, the two subjects being very properly studied together after the courses are completed in mathematics and physics. Power and power transmission, prime movers, and electrical engineering will be taught in the third year, all students taking the same amount, no matter what their future specialty. For the civil engineers, however, the subjects will embrace a minimum of design, special attention being paid to the subjects of selection, cost, and installation. No great amount of drafting will be done in this year, not more than is absolutely necessary, but throughout the year, as in the preceding ones, the atmosphere must be that of the office rather than of the school.

Instruction in English will occupy more time this year, and much practice should be given in the reading and interpretation of specifications and in searching for faulty phraseology. A study should be made of the common errors of speech and their correction. Each week the dean should call the attention of the students to leading articles in the best technical periodicals, and the professors in each subject should quiz the students in order to ascertain that certain articles have been read.

Instruction should be given in public speaking, and all students should be compelled to attend not less than six meetings of the city council of the nearest incorporated city and learn how to conduct themselves later in life when occupying positions of responsibility and trust in the public service. The meetings of the engineering society of the school should generally be devoted to topical discussion; the discussions should be arranged by the professors, and certain students should be given subjects to look up and prepare papers upon, to be read at the meetings. If any local engineering societies meet near enough to the school for the students to be able to attend, advantage should be taken of this to get the boys in touch with men in active practice. For all such work college credit should be given. Psychology should be a required subject this year, and the students should also have plenty of electives from which to make a choice, the studies mentioned being the only ones required.

In the fourth year the required subjects will be hydraulics, political economy (with special reference to finance, banking, and the labor movement), sociology, and hygiene. The remainder of the subjects should be technical and elective entirely, as each student, by this time, will have a preference for some branch of engineering. This preference may be natural or it may have been gained during his vacation work, provided he took any vacations.

Training such as outlined should produce young men who know about as much physics, chemistry, and mathematics as will be found necessary in actual practice. They will be far better assistants, in the minor positions to which young men are assigned, than the average graduate of to-day from American schools. The training will be a trifle harder than at present, but the student can see the utility of every subject as he studies it. He takes nothing for granted, but realizes at every step of his progress through school that he is really learning something that will be of value to him in his work. Those who have a fancy for some particular subject will have an opportunity in the last two years to go as deeply into it, as an elective, as they may desire, while those whose bent of mind is of a more business-like nature will read technical books under the guidance of their professors. Reading, research, and library work should play a great part in the entire course, considerable topical work being given. The elective studies in the last two years will take care of some subjects a few readers may consider should not have been omitted. Taking the course as outlined, however, and omitting electives, the graduate should be very useful immediately after graduation in the office of any engineer, no matter what his specialty, and his specialty has a better chance of selecting him, than if he is too narrowly trained. The product of such a school should be educated so that "He will be on to his job and can make good"—a most desirable thing. The difference, as compared with very many of our schools at present, will lie in the cultivation of an "office" atmosphere rather than an academic one in the halls of our engineering schools.

The thesis work now requiring so much time in the senior year can be abandoned with considerable advantage. Professors forget that the students at an engineering school, after all, are only boys, and the endeavor to have them show "ability to do original work" is rather absurd. A certificate upon the completion of the engineering course is all that should be necessary in the way of a diploma. If a degree is considered essential, no harm can be done in granting that of Bachelor of something or other. Thesis work, however, should be left for those few students who remain, or return, for graduate work.

The writer perhaps may not be a fair judge on this detail of the thesis requirements, because, as a contractor's engineer, he had the bad fortune to have to construct one water-works system and two sewer systems in medium-sized cities, which systems were the subjects of the graduating theses of bright sons of those cities. Their professors were the consulting engineers, and, when controversies arose, managed in all cases to escape deciding anything. The plans were very amateurish, and the professors had obligingly permitted the use of their names to help impress the city officials.

The "capacity to do original work" is not proven by the preparation of a thesis, and a great deal of valuable time is wasted and needless expense incurred in the preparation of essays which are filed away in the archives of the colleges, never to be brought to light. The thesis work is the one attempt on the part of the teacher to obtain an office atmosphere, but the result is generally an intensifying of academic influences. A few men will return for graduate work, and they should be put at research work; but this is all a matter of detail and apart from the proper training of competent assistants for engineers. It is university work, as distinct from college and technical-school work.

DISCUSSION

Mr. Garver. N. B. Garver, Assoc. M. Am. Soc. C. E. (by letter).—This paper has been read with much interest. Many of the statements made therein the writer believes to be true, but there are some things which cannot be accepted without a grain of salt. For example, the author advocates a school year of four terms of 12 weeks each, the student being enrolled in any subject of his course in any one of the four terms of the school year. This means that every subject required in a given course must be taught during every term, a condition which is impossible to realize in practically all engineering colleges, unless the instructional staff is doubled or trebled, and unless the number of classrooms at the disposal of the teaching force is materially increased. Few of the engineering colleges have the financial backing to permit this.

Again, the author advocates an idea which is not considered practicable in engineering construction work: He thinks the practicing engineer should specify the course of instruction and the way it should be imparted, and that the builder—the school—should guarantee the quality of the product. To quote:

"He is a stronger man who adds to his natural ability the power given him by the pursuit of a carefully arranged course of study in the essentials of engineering science at a well-conducted school. This being the case, it should be settled by engineers, and not teachers, just what instruction is best and how it should be imparted."

Again:

"Manufacturers of materials used by engineers have been compelled by competition to furnish products satisfactory to the user, so why should not the schools, that manufacture embryo engineers?"

The writer agrees with the author that the more nearly the atmosphere of the college of engineering approaches that of the office, and not that of the school, the better it will be for the student. The difficulty with a great amount of the work done by the student is that he does not see any direct connection between the work he is doing and the practice of engineering. Much of his work could be arranged so that it would be done in practically the same way as office work. Too many students acquire the habit of doing things in a happy-go-lucky sort of way, working when they feel like it, or spending their time in social functions, rather than in some work that will advance them in their chosen line. Most students should be placed in such a position that it would be necessary for them to develop a systematic way of using their time and also a systematic way of doing things. It has been observed by the writer, on more than one occasion, that the student whose time is fully occupied, and who must necessarily use system, does work of a better grade than one whose time is not fully occupied.

Mr. Garver.

The young man who completes an engineering course in a college is not an engineer, and employers should not expect such in engaging technical graduates. The writer does not believe it is within the province of the engineering college to teach men to be expert draftsmen or what-not. A good letterer is produced by practice and not by college training. The practicing can be done as well outside of college as in it. The engineering college should give to the student, during the four years he is in attendance, the things he cannot gain through his own efforts. The facility with which a student may get a job and hold it is not the only thing to be kept in mind in engineering education.

Mr. Pillsbury.

George B. Pillsbury, Assoc. M. Am. Soc. C. E. (by letter).—The remarks and recommendations of the author seem to indicate an opinion that the technical schools should advance yet farther in their tendency toward practical training. The writer must confess to the view that both the technical and the secondary schools have already advanced too far in this direction.

What, indeed, is the object of the technical schools? The author regards it as the training of young men to be proper engineering assistants, who may develop, if they have the opportunity, into successful engineers. The writer would suggest that the object should be rather to prepare the students, so that in due time, if they are able, they may become successful engineers.

If the object of the technical schools were the training of assistants, the writer would be inclined to join those who decry the schools and all their works. For who could recommend the expenditure of time and money necessary to complete a course at a technical school, if the end were the preparation of the student as draftsman or instrument man? The concentration and discipline of an office or a field party will teach these duties in far less time and at far less cost; nor can the schools hope to turn out men whose skill and knowledge of details is sufficient to make them immediately proper engineering assistants. The Profession is of far too wide a scope, and is divided into far too many branches, to make such an ideal possible.

It would appear then, that the function of the school should be the teaching of those things which are not taught in the field and office. They should teach the fundamental "whys" of the Profession, rather than afford a smattering of the infinite "hows." More than this, they should strive to discipline the minds of the students into that clear thinking which leads a man to view his task in all its bearings, so that it may be done wisely, in conformity with the spirit of his instructions, and not mechanically, in conformity with the words.

It will be generally admitted that, as a means of mental discipline, nothing can equal a rigorous course in pure mathematics. A "practical" course only encourages the habit of loose thinking; and, if the mind be properly trained, the practical applications are easily

Mr. Pillsbury. acquired. The course should not be so long, it is needless to say, as to interfere with the sciences, and with the theory of the mechanics and the materials of engineering, but it should be rigorous, and not "practical."

No disparagement of the so-called culture courses is intended. These studies, if really taught, not administered as sugar-coated pills, must give the student a broader view of life, to his material advantage.

A real knowledge of written French and German opens sources of information which are not to be despised.

Practical instruction must be given, to maintain the interest of the students; but as such instruction, at a school, is essentially impractical, it would appear that it should be definitely regarded as a means and not the end. It is suggested that the need of the schools is more concentrated effort on the part of the students, rather than a reduction of the academic side of the curriculum.

Mr. Constant. F. H. Constant, M. Am. Soc. C. E. (by letter).—Every engineering instructor who reads Mr. McCullough's paper will appreciate the justness of many of his criticisms, and will be deeply interested in his suggestions for the improvement of the product of our engineering schools. That this product is not wholly what it should be, is a statement which need not be debated. It is of more immediate concern to learn wherein lies the explanation, and how the conditions may be remedied.

The technical schools are confronted with the following conditions: The grist that comes to their mill is of a more varied kind and in more overwhelming quantity than ever before. Every parent, whether well-to-do or not, seeks to send his children to college, and in these days of wide-spread and cheap educational opportunities, this is not difficult to do. There is a general turning away from all manual and artisan occupations toward professional and business careers. Engineering, which formerly was a little known and somewhat contemptuously regarded vocation by those outside its ranks, has leaped into a well-deserved popularity. It is now regarded as a fine, manly and, withal, genteel profession, full of interesting possibilities, appealing to the constructive instincts of a man, and developing his best mental and moral forces. There is also a lurking impression that it is a lucrative calling.

The result is the hundreds of young men who knock annually at the doors of the engineering colleges. Many of these have little or no conception of the nature of the profession they seek to enter; some are by nature unsuited; and not a few are there only at the earnest solicitation of parents who know little about engineering but think it is a fine profession for their son. Nearly all instructors lament the fact that the average present-day student lacks the earnestness, singleness of purpose, and interest in his work that characterized the smaller

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classes of a few years ago, and this is probably true. Formerly, few entered an engineering college who did not know what they were there for and how to get it. Engineering was not then so complex; the need was for men trained and expert in technical details; and this kind of training the college is eminently qualified to give.

The trouble is that the engineering college has not fully recognized the changed conditions, and has sought to patch up the old machine to meet present-day needs. Instead, a very different raw material comes to its hopper; the internal treatment needs to be modified correspondingly; the mechanism for eliminating the unsuitable requires to be made more effective; while the final product itself should be of a somewhat different character than formerly. This is the situation which confronts the engineering college of to-day.

The first and most natural effort of the engineering college is to try to reproduce the old conditions, with which it is already familiar, by raising the bars of admission. That there should be a radical reassortment of the material which seeks to enter the Engineering Profession is apparent to all, both inside and outside of the college. The cogs of the most perfect machine cannot run smoothly when it is called upon to digest too large a proportion of unfit material. The country, however, needs—much more than it needs more young engineers—men who are skilled in the use of their hands as well as minds, trained artisans and handcraftsmen. It is time to rid ourselves of the self-satisfactory delusion that the American workman is the most clever craftsman in the world. This may be true along some narrow lines, but, in general, we are unintelligent and inefficient in the combined use of hand and brain. One has but to pick up any article or device requiring a little intelligent craftsmanship to find, stamped somewhere upon it, "Made in Germany." Why should it be necessary to import from some other country everything requiring taste, thought, and manual skill in its production? We are not the most clever people, but, at the same time, we are not the most stupid. Our ranks are annually recruited from the artisans of these very countries which thus outstrip us. We need more schools, and many of them, for the training of skilled artisans. We seem, in our haste to get the material goods of this world without working for them, to be developing a nation of business men at the sacrifice of trained workers. The result is restlessness, discontent among all classes, a depreciation of the skilled trades, and the high cost of living. The establishment of the higher trade or craftsmanship schools would deflect away many who now enter the engineering colleges, to the mutual advantage of both.

Probably nearly every instructor, who theoretically favors lowering the bars of admission, rather hesitates to see it put into operation, through an instinctive distrust of the efficiency of the processes of

Mr. Constant. elimination within the college. The judicial side of the instructor's work is the most difficult and distasteful, requiring acumen, experience, moral courage, and a true knowledge of correct standards. He is assailed, on the one hand, by every pressure which the weak or lazy student can bring to bear, while, on the other, the judgment of the profession at large is indefinite and somewhat remote, and is only felt after cumulative years of experience with the product of the college. The line of least resistance (and often of inclination) is to yield to the student. That such a large number of unfit men is actually eliminated from college, indicates that the average instructor is conscientiously "on his job." That a further improvement is needed, is apparent to all. With a greater perfection in the mechanism for ejection, the entrance bars may well be lowered, as suggested by the author.

After his entrance, the first work of the college is to care for the student and properly prepare him for his life's work. The author has suggested a curriculum which will probably not elicit a great amount of criticism. The details and arrangement of the curriculum may, perhaps, be confidently left to the colleges to develop. Most of the technical professors in the leading engineering colleges have been and are practicing engineers, while the younger men teaching the same subjects are generally drawn directly from the ranks of the Profession. It may be assumed, therefore, that the engineering college is in close enough touch with the active profession to understand the nature and quality of the work it should perform. Nevertheless, every member of the teaching profession will welcome constructive suggestions like those presented by the author.

After all, however, it is not so much the precise nature of the curriculum as the manner in which the subjects and the students are handled that is important. How to bring out the very best in every man, to stimulate his interest and devotion to his work, and, at the same time, to eliminate the lifeless and the small group of deficients always to be found at the lower limit, who, by sheer persistence, in point of time, finally get through, no more fit, perhaps, at the end than at the beginning—this is the real problem of the engineering school.

The author has suggested that the office atmosphere should early be introduced into the classroom, and with this the writer is in full agreement. It is somewhat incongruous, to say the least, that a young man of about the age of twenty should be handled by the same academic methods as those used in the high school, and that he should be led to believe that success is measured by the attainment of a certain grade mark, whether he really knows anything about the subject or not, or by the mere making of certain design drawings or laboratory experiments, whether accurate or not, or whether largely inspired by others or not, when immediately he steps out of the college door into the real world of his calling, for which he is supposed to be fitted, he finds that he is judged by entirely different standards—not by what he pretends to know, but by what he actually knows; not by what he knows, but by what he can efficiently use; not merely by what he knows, but by what he does not know, but knows where and how to get it; not by what he can use 70% accurately and 30% inaccurately, but by that of which he has a command of 100%—in a word, by his accuracy, real efficiency, self-reliance, and vitality as a growing and expanding force. If the college cannot even suggest to him what is in store for him when he leaves its walls, it has failed to live up to the measure of its opportunity. Mr. Constant.

Probably there is little room for change in teaching methods in the earlier subjects of the curriculum, such as mathematics, physics, etc., except that even these should always be taught with the future needs of the engineer in view. A good deal of fine judgment in the elimination of the unfit should be exercised. A large part of the weeding out of the unfit must be done in these earlier years, both in justice to these men and to keep the wheels of the machine from becoming clogged. Men should be judged, not solely by their marks, but by their real proficiency in each subject; and the unfit and lazy should be ruthlessly and firmly ejected. Even the professor of mathematics should constantly seek to discover those qualities which promise to make future good engineers. For the product of the engineering college is not mathematicians but engineers. The professor's work, from the beginning, should smack of office and field, and accuracy and efficiency should be striven after, rather than a great range of knowledge.

In the higher technical work, especially that of designing, the office atmosphere may be exactly reproduced, and the men put upon their mettle and judged by the same standards which they will meet when they leave college. In this way there will be no abrupt transition between the life of the college and the working world; and this, if the writer is not mistaken, is what the author and all other employers of young engineers desire. The creation of the office atmosphere and the reproduction of professional conditions is the problem which the instructor must solve, and which, to a large extent, depends on his own personality and experience. In engineering, as in every other department of learning, the college is not the walls, classrooms, or equipment, but the men who teach; and the good teacher is just about as valuable as the good engineer.

The writer does not hereby advocate the introduction of all the office details and routine. The student's time in college, and in the technical work, especially, is too short to admit of this, even if desirable. He has, for instance, never believed that it is the function of

Mr. Constant. the engineering college to train men for the drafting-room of the manufacturer. The latter is usually willing to pay promising graduates to learn to make shop drawings. What the employer does insist on is that the graduate shall be promising, and will take hold promptly and in the right spirit. It is the duty of the college to instill the spirit of the office rather than introduce all its details. It must also look ahead and train for next year as well as for to-morrow; to start men in the direction of becoming good engineers, as well as good draftsmen.

What are the elements of the office which the instructor may introduce into the classroom? They are: the individual problem and responsibility for the same, self-reliance, accuracy, and efficiency. In regard to the first: when several men are put on the same problem, each is very prone to lean on the others, and it often happens that one strong man does all the thinking for a group of weaker men. The individual problem, which may be a definite part of a larger problem, is the condition actually met in the office. The student is then thrown wholly on his own responsibility. Of course, this places more work on the instructing staff, and is only possible for small sections, say ten or twelve at the most.

In regard to the second—self-reliance—the student should be made to get by himself, as much as possible, the technical knowledge needed to tide him over the difficulties and solution of his problem; and to go to the instructor only in matters requiring judgment and experience. The latter, of course, will watch and check him constantly, and not let him go too far astray. He will always seek to develop the student's self-reliance and ability to meet his own difficulties. Some of these difficulties, and the larger aspects of the problem will be discussed by instructor and students in the frequent get-together periods, commonly called seminars.

Finally, the student will be held to a certain degree of proficient achievement and accuracy. His work will be turned over to another student, perhaps in another section, to be checked completely, and finally to the instructor; who will have his own complete notes. This is probably very similar to Dean Raymond's plan. The writer will have an opportunity to put it into operation in his own department during this coming year.

It is hoped that this plan will stimulate the interest and develop the best powers of the students. The instructor must stand ready to judge his men as impersonally and critically as does the chief draftsman or engineer employer, and to insist on a high grade of work, after making due allowance for beginners. It is impossible to grade a student, with this plan, by marks or examinations, but solely by the quality of his work from day to day, and the way in which he takes hold of it. When the student shows a grasp of the subject and promise

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of future proficiency, but is slow and falls behind in quantity of work executed during the term, he should, without being marked down, simply be required to spend a longer time on his task. One of the evils inherited from the academic college is the class system, which assumes that all men are equally quick in perception and performance. The writer is inclined to agree with the author that the engineering college might well utilize all the months of the year, or, at any rate, that every subject should start afresh with each term, thus giving the student the opportunity to drop out and re-enter at any term point, as he may wish. The student who does not complete his work in one semester may continue with it into the next, getting his credit when he has finally satisfied his instructor that he has acquired the requisite proficiency in the subject. In like manner, he gets his certificate when he has finally completed the course, whether it is in three or in five years.

Another strong reason for this greater flexibility in student movement is that it readily permits him to acquire practical experience in shop, field, or engineer's office during his course. In the German technical schools, in all but the *"bau construktionen"* courses, twelve months in some industrial shop or laboratory is required before the student is given his certificate of *"diplom-ingenieur."* If the object of the engineering college is to train men for engineering work, and this is to be done by placing them as early as possible in the engineering atmosphere, it is a distinct aid to the teacher to have his students go forth and find out by actual contact what this atmosphere is like. Every instructor knows that the men who have thus interjected a year or two of practical experience into the middle of their course return more earnest, full of purpose, and desirous of making the rest of their time in college count for the most, than if they had had no such arousing experience. Dean Schneider is accomplishing this in one way, at Cincinnati, and the Germans in another, by actually forcing their men to get this outside experience, and all colleges should make it fairly easy for their men to drop out occasionally, if they so will. Those who are tempted to remain out permanently either will have good reason for doing so, or, as the author suggests, are such as the college and the Profession can afford to lose.

Summing up the foregoing points:

1st.—The engineering colleges are confronted with new conditions, not of their own making, but due to the economic and social trend of the times.

2d.—The conditions can be controlled somewhat from without by diverting some of the stream of young men now entering the engineering colleges into artisan and craftsmanship schools, and by elevating, in the public estimation, the dignity of such manual occupations.

Mr. Constant. 3d.—After such a natural sifting, the colleges might well lower their entrance requirements, so as to make it reasonably certain that no earnest and promising young man is excluded.

4th.—The college should seek to train men for engineering work such as they will actually find it when they leave its doors. This can be done by a proper choice of curriculum, by the selection of wise teachers, experienced both in the practical side of their specialty and in the best means of presenting it to meet the needs of their students, the latter through the creation of the office atmosphere, at least in the upper technical work.

5th.—The processes of elimination of the unfit should be effective and rigid. Only such as an employer would care to retain and develop should be permitted to graduate.

Finally.—Practicing engineers and manufacturers should be willing to co-operate with the colleges, as far as possible. This point, however, need not be emphasized, for it is the general experience of the colleges that the former are in sympathy with their work, and co-operate cheerfully whenever a call is made upon them.

Mr. Green. Arthur B. Green, Jun. Am. Soc. C. E. (by letter).—For the purpose which Mr. McCullough has in mind, in educating future engineers, his curriculum seems well designed. If a man who is "on to his job and can make good" must be accepted as an educated man, certainly the limitations of Mr. McCullough's discussion of education are justified, and he has completely solved the problem which arises out of that assumption. He has shown how to "manufacture," as he terms it, a perfect assistant. His product would contain exactly the right ingredients of knowledge, proportioned according to exactly the right formula, and, above all, every safeguard would be provided that it might contain nothing superfluous. It would be an accurately standardized article. Each employer would feel the utmost security that, from the very first day, this ideal assistant would return to him the full value of its price, and if, in any case, there were doubt about it, the employer could, possibly, turn it over to the United States Bureau of Standards for quantitative analysis, and have its ingredients properly checked. Standard specifications are actually called for, to be prepared by employing engineers and handed to the schools, and the schools may be put on contract to furnish only product fulfilling these specifications. Indeed, when employers begin paying the expenses of these "engineer factories," perhaps this ideal can be realized.

It would be too bad to take any of this very seriously. It is not enough that a professional man shall be "on to his job and make good." Nothing more than that is required of skilled laborers or trade mechanics; the engineer must be educated—school-educated, self-educated, or, preferably, both. Education, according to Webster, is defined thus:

"The totality of the qualities acquired through individual instruction and social training, which further the happiness, efficiency, and capacity for social service of the educated." Mr. Green.

Qualities make up education, not knowledge. Moreover, these qualities must "further" certain things about the educated, so that education is not a possession that is complete and perfect according to any specifications, but it is to grow and advance. Inasmuch as so many discussions of engineering education have gone wrong with their conceptions of the subject, these two points ought to be taken up more fully, to see what are the consequences.

In the first place, then, knowledge is not education. Just what subjects are studied by the one being educated is a secondary matter; the chief concern is that the study shall be inspired and directed in such a way as to develop qualities which further happiness, efficiency, and capacity for social service. Many great engineers there have been, and there are, who never studied a technical subject in school, and one electrical engineer of great note received all his schooling in literature and the classics. He is educated, nevertheless. He has the qualities of education at such potential that they are to be measured almost by the kilowatt. It is the function of technical schools, no less than of universities, so far as they can, to help along the development of such men as this. Their prime duty is to educate. Should they fail in that, the excellence of their curricula could not redeem them.

In insisting, therefore, on this distinction between education, on the one hand, and instruction, on the other, there is no quibble. On the contrary, it is definitely harmful to confuse the two. In an employer looking over a candidate for employment it means laying the emphasis on just the wrong thing, and considering what he has, instead of the power he has to obtain more. From the business standpoint, it means reckoning immediate profits as more important than ultimate strength, both for the individual employer and for the Profession at large; and it also means regarding the assistant not as a future engineer but as a piece of equipment. Mr. McCullough has been more frank than most others who take this attitude, and just as negligent of the consequences. The moment that the assistant to the engineer becomes standardized as a piece of apparatus for specific uses, immediately he is a trade mechanic and not an engineer. All he has to sell are his contents. He is designed to fit specific needs, and finds place only where those needs exist. Always working at the same manner of problem, he is unfitted to undertake anything new or to advance; his price is soon fixed accordingly, and he is a commodity. He has then entered, not the Engineering Profession, but the Engineering Trade. There is such a trade. Its members are somewhat learned and considerably skilled, but hardly educated. It is unfair to the engineering recruit to require him to fit

Mr. Green. himself for this trade and no more, to put him in competition with those who devote their lives to the trade, and to hazard his future on the outcome. It is unfair to the Engineering Profession thus to force its more highly endowed and better trained recruits into this trade, and so, temporarily at least, out of the Profession. That is a very bad use of opportunities.

From what has been said so far, it may be gathered that the intention is to underrate the value of special knowledge in engineering. Possibly so. Yet, is this not true: Wide knowledge as well as the qualities of education are required that a man may do more than be "on to his job and make good"? One of the most absurd mistakes is to presume that there is any harm in the student learning beyond his needs. Learning gives power to learn. The value of it is not so much the storage in the mind of certain information as the development of power. It is not undesirable, but absolutely essential, that the well-trained student be guided, while he may be, carefully in problems with which the world would not trust him for years after. He will be ready for them, or others like them, at the earliest possible date, and therein is strength for the Profession.

Secondly, and lastly, education is a process of growing, not a fixed attainment. If engineers are to employ men with a start toward education, they will not get the finished product. The work of finishing has only begun, and must continue. Who, then, is to carry it on? Employer and employed must carry it on. Sometimes it seems as if employers were trying to dodge this elementary idea of growth in young engineers purposely, in order to escape the necessity of doing anything toward it themselves. They try to unload upon the schools the whole responsibility for the education of coming engineers, and to assume that, once they have told the schools what they want, they have washed their hands of the matter.

It takes only a moment's reflection to see that this is not only an unpractical stand, but otherwise a harmful and absurd one. The burden of the education of young engineers is not on the schools at all, but on the employers. They cannot escape it, if engineering is to be a Profession. It is the highest duty of every employer to study his assistants, to discover their capacities and adaptabilities, and to develop to the fullest every potentiality. This is the highest duty, because the employer owes it, not only to the assistant and to himself, but still more to the Profession at large, and most of all to the public. Both the Profession and the public benefit vitally by having the best engineers for the world's engineering. The employer is the one to furnish them.

There are many employers who recognize this principle, and still attempt to escape the duty it imposes. A decided example is the head of a large contracting business, whose work from office to shoveller is most thoroughly organized. He recognizes it as a fundamental principle of scientific management that the employers must teach the employed the very best way of doing each single operation on the work, not only once but continuously; that the employers must learn to correlate the capacities and adaptabilities of the workmen with the requirements of the work so as to choose scientifically the best men for each job. Thus he spends large sums and much energy in picking out promising bricklayers, teaching them the best way to lay bricks, seeing that they do lay bricks in that best way, keeping account of their performance, and tracing thoroughly and rewarding the advancement of each individual bricklayer. He does the same for helpers, for those who load and unload his materials, for his office force, and so on. He looks upon all this as the duty, the extremely profitable duty, of the employer. He is right; but he omits his engineers. It is one of his axioms that fresh technical graduates are of no use; therefore he will have nothing to do with them until they have had three years or so of experience. In other words, his competitors may train them for him. Although he is ahead of his time in insisting on the harm that stand would do him if taken in regard to bricklayers, he is oblivious to the harm it does him in regard to engineers; but the real damage is greater than that. It reaches to the recruits as well as to the contractor, to the Profession more than to the job, and most of all to the public. Mr. Green.

It is idle for any engineering employers to seek, as does Mr. McCullough, to unload upon the schools the whole burden of engineering education, for that belongs chiefly upon the employers themselves. Rather than study to appreciate the shortcomings of technical graduates, it would be fairer and more profitable to try their possibilities. When that is done, employers and teachers will become mutually helpful, as they ought to be, and a wise solution of the problem of engineering education may be surprisingly near at hand.

Alexis Saurbrey, Assoc. M. Am. Soc. C. E. (by letter).—It is very important to distinguish between "Engineering Education" and "Engineering Training." As to the first, education is, or should be, the common property of all civilized men, and the engineering school should not waste its time on the hopeless task of instilling true education, where home, environment, associates, and natural disposition have failed. Schools, colleges, and universities are struggling in vain when they attempt to "teach" taste, good manners, and gentlemanly behavior, if these qualities are not planted in the average boy at home, or, in many cases, acquired by less happy boys through natural disposition. "The well-read man is generally able to pose as a 'cultured' man," Mr. McCullough will have us believe. The writer denies this proposition, as well as the desirability of teaching young engineers to "pose." Certainly, it is a pleasure to meet a cultured, well-balanced, considerate man, and we cannot have too many engineers of that kind; Mr. Saurbrey.

Mr. Saurbrey. but if an engineer is not so well-equipped, let him by all means avoid the deceit and shame of "posing."

The writer, therefore, thinks that the sole problem of the college is to train. It cannot hope to train for the exceptional position at the top of the Profession, but it can, and should, train for usefulness in the common, average case. The young engineer leaving college should be able to do correctly what Mr. McCullough properly refers to as the clerical work of engineering: Compute quantities, calculate stresses and strains, use the level, tape, and transit, and so forth. When, as a matter of fact, he cannot do that, the colleges are not solely to blame, for the preparatory school should have taught precision in algebra and arithmetic, which it does not do. Any really efficient reform movement in engineering education must begin with the home, and must fully consider the public school. With this attended to, the college will automatically adjust itself, and produce better engineers from the better raw material.

Nevertheless, some of the criticisms of the colleges are justified. No doubt the very broad training leads to neglect of details, and to superficial study. The remedy lies in an extension of the time for the purely engineering training, and in a curtailment of the volume taught, especially a reduction in the introductory studies of the first two years, whereby more time might be gained for the real engineering subjects. Such items as chemistry, physics, descriptive geometry, geology, and higher mathematics might, profitably, be reduced in volume, with the proviso that the subjects taught be really and thoroughly assimilated by the student, especially the simpler problems in analytic geometry and calculus.

The course in civil engineering, properly speaking, should certainly not be less than 2½ years (better 3 years), after the completion of the introductory studies referred to. All this time should be devoted to a most thorough drilling in fundamentals, with very little attention to generalities. The use of mathematics should be reduced to an absolute minimum, all complications being carefully avoided; understanding should be the goal aimed at, that is, intelligent application of thoroughly understood principles. Only a very few branches of civil engineering are on a truly scientific basis, and this fact might be taken advantage of, and engineering taught rather as an empirical profession than as a science; in other words, do not bother too much with the mathematical proofs of propositions which are, in reality, proved only by experience and experiment. The impossibility of transmitting telegrams across the Atlantic, the impossibility of flying, have been proved time and again mathematically, and yet the possibility was proved the next day in practice.

Without doubt, many teachers are trying to do just what is suggested here, and, if so, the writer feels that they are on the right track, Mr. Saurbrey.

and wishes that they would go still further. Many colleges, also, during the last few years, have given additional attention to the commercial side of the question, and correctly so. While the writer certainly would be the last to excuse rank commercialism in anybody, he recognizes the fact that the engineer's principal purpose as an engineer is that of increasing values with as little expenditure as possible. The engineer is a wheel in a great commercial machine; as soon as he emerges from the modest initial incubator stage, he deals almost exclusively with business men; and the one question he has to answer is "what does it cost?" If, in addition, he cannot show that he himself is a fairly good investment, he will assuredly lose his job to the one who can. As it is, it takes a good while for the young engineer to satisfy himself and others that he is really worth his salary, and that is not right. It will be different when the graduate has been taught the immediately useful facts and formulas, and when he has ability to discriminate between extravagant and economical design of simple structures.

It is not necessary to state that the college should teach its students the rudiments of bookkeeping and cost keeping. Instead, it seems that scientific management has been taken up. If hereby is meant "motion study" and such matters, incalculable damage will be done, for men are not machines, and should not be treated as such. Moreover, the writer believes that this fad will be a thing of the past in a few years, and the college should be very conservative in introducing such matters.

Mr. McCullough's paper, as well as his recent book "Engineering as a Vocation," are most valuable and interesting. They disclose in a clear, concise, and wholly unprejudiced manner the very foundation for that dissatisfaction so common among recent graduates, and so often expressed by them in the engineering press. It is not only a question of pay, for engineers are as well paid as attorneys and doctors, and much better than teachers or ministers, all of whom have to put as much time on their training. It is mainly a question of competency, of ability to render service in the world as it is—the engineer seeing the great opportunity he has for service while the public does not; but the public will. The engineer of to-day is a pioneer who must clear the forest of misunderstanding, indifference, and inertia, and that takes time. In addition, the fields opened by the modern testing machine, indeed, by the modern spirit of research, have not been properly explored, and we still suffer from many "ifs" and "buts" to be solved in the future. The problem of writing good text-books is no easy one, when new research makes old truth obsolete over night, and, as long as the teacher must study the changes in the fundamental theory, he is greatly handicapped as a teacher. For this very reason, reading knowledge of foreign languages is almost indispen-

Mr. Saurbrey. sable to an engineer who wishes to be up to date in his specialty; but they should be taught in the preparatory school, and along practical lines, not in the college.

On the surface, the problem raised by Mr. McCullough seems possible of satisfactory solution; but in reality it is one closely connected with the home and the public school, and, therefore, with the community at large. The battle-cry of to-day is reform, the enthusiasm behind the guns is dissatisfaction. One question, indeed, suggests itself strongly: Is not the failure of the weak, and the survival of the fittest, a principle against which we are battling in vain? one that will exist even if the most ideal vocational training were given? Surely those who are now dissatisfied engineers would otherwise be dissatisfied mechanics, and no happier than they are at present.

Mr. Cohen. J. X. Cohen, Jun. Am. Soc. C. E. (by letter).—The author aims in the proper direction. He seeks to serve the student first and then his future employer. The sound, fundamental, non-specialized technical course which the author recommends makes the student broad and receptive, rather than narrow and exclusive.

It is encouraging to note that the course outlined emphasizes so greatly the study and the value of English. By English is not meant the polished literary language of the library, but the sturdy style of the council chamber and the business office. To the great detriment of the engineer, his English course has usually been made a minor one, and very often neglected at that. That is a very serious situation, and calls for rapid remedial measures. Certainly, engineers should first know how to handle materials, but what more valuable materials are there than men, and what means of communication between men exists, other than language? Even when engineers deal with each other directly, what matters it how well their minds may operate if the thoughts cannot be transferred clearly and correctly? We all know men who have good ideas and excellent thoughts which are hardly ever realized, solely because they are not plainly stated. The ultimate significance of the idea cannot be quickly made clear to others, and it dies before it develops.

The author considers that course in engineering most beneficial which permits of alternation between class-room and field, between school and shop. The writer, having received such a training, and having further observed the comparative effects of the older method of training, heartily endorses the newer.

There are several technical high schools in New York City, the graduates of which are equipped for entering either the engineering school for advanced studies or the engineering office for practical work. It may be of interest to state that a very large percentage of these graduates goes immediately into actual work rather than into college, without, however, having abandoned the idea of a higher technical education. Mr. Cohen.

Having secured a position which their technical high-school training qualifies them to hold, they next enroll in the evening engineering course of the Cooper Union for the Advancement of Science and Art, or some similar institution, of which there are also several in New York City. Here they spend their evenings for a good many years—five years at Cooper Union—in hard, arduous, and comprehensive study, supplementing the practice followed during the day with the knowledge gained at night.

This method of study makes for the greatest good. The co-ordination of class and field produces results which are harmonious and well-balanced. Studies are pursued with the greatest interest; their immediate application in practice is either actual or plainly discernible, and their utility needs no emphasis by the instructor. Very often the problems arising during the day may be worked out in the laboratory or class-room during the evening. This produces impressions which are vivid and knowledge which is secured. At times the pace in the class-room would appear to the regular day school instructor to be extraordinary. This combination method makes speedy and successful studying possible.

Such a combination course helps a man financially in several ways. For one thing, he is self-supporting throughout all the period of study, despite the fact that such a course may take a longer time than the so-called regular one. He is employed constantly, and not only during school vacations. This surmounting of the financial barrier is valuable to the Profession, for otherwise many good men would find it hard to prepare properly for practice. For another thing, the combined day worker and evening student finds that as his technical knowledge increases his employers correspondingly increase his compensation. As he observes his increasing pay, he notes the effect of his spare-time study on it, and, as a result, the incentive for further and more concentrated study is greatly strengthened. Better than a good report card is a larger pay check, for while the first predestines the other as an eventuality, the second is the actuality. Not all men, especially in engineering, work for gain, but the stimulating and encouraging influence of tangible recognition is highly beneficial. Finally, a man is helped financially—as well as in numerous other ways—by being kept so busy that he finds no time to get into mischief.

The graduate of the combination course, when he receives his degree, is handed a certificate which shows that he has demonstrated his capacity for hard, continuous, single-centered work. If he had not possessed this ability at the beginning of the course, he would never have reached its end, except through the inculcation of that faculty

Mr. Cohen. in him by the example of his fellow-students. If for nothing else, such a course is of value as a demonstration of the true capability of the man to do diligent work and his real capacity for conscientious, continual toil. Too few men realize until very late in life the enormous amount of work that can be accomplished without undue fatigue by strict adherence to a carefully planned programme. Further, the utilization of spare time for self-improvement is taught in an unforgettable manner, and as the graduate must necessarily be a student after graduation, by pursuing the combination course he learns how and when and what to study after his college days are over. The waste of spare time prevalent among many young engineers is great, and it is a waste which is a direct result of the lack of early training in spare-time study. The student of Cooper Union learns to work even when traveling on trains, unconsciously following the example of the most eminent consulting engineers in active practice. He who learns how to utilize all his available time efficiently has a splendid start in the race toward professional success, which ordinarily can only be attained by continual concentrated application; and to this type of application the graduate of the combination course is no longer a stranger.

The student who is engaged simultaneously in the study of engineering and its practice enjoys a great privilege. He can ascertain whether he has that aptitude and inclination for engineering, which, to a great extent, is vital to success long before he has invested much money in his course or much of the even more valuable time in its study. He has the advantage of being able to decide whether engineering appeals to him as a life work at a much earlier stage than the regular school student. The number of students who are graduated from the regular course, and fitted by training for engineers, is now very large, but of these only a fair percentage is fitted for it by natural talent, inclination, and equipment. Many realize this some years after graduation, but then it is too late, from their viewpoint. Having spent so many years in preparation, they fear to see all their efforts go to apparent waste. They also greatly fear the possible ridicule of their friends at their early recognition of and submission to failure in their chosen calling. Such motives as these keep many men in the ranks until, by force of circumstances, they are forced out or forced up. For a long time, however, they encumber the lower rungs of the ladder, making it harder and harder for themselves as their numbers grow, and also more difficult for the young engineer of future merit to obtain a foothold; but whether or not they stay in the Profession, they have suffered a grave economic loss. In this loss the community at large is also a participant, and it is to relieve the public and the prospective engineering student from as large a measure as is possible of this partly preventable loss that the combination course is advocated by the writer. Mr. Cohen.

The Profession is benefited directly by the combination course. Few but the strong, the steady, and the persistent complete such a course, so that the process of weeding out starts at once and has just that much longer to operate. It is an effective block to the lazy, unambitious young man, who would stand but a slight chance were he to enter active practice. If time were available, the writer would like to discuss the role of the engineering teacher in the school attended by students who are at the same time in active practice, but suffice it to say that these teachers must be mentally alert, on the very *qui vive* for the latest and best information and methods of its presentation, and altogether on a high plane, in order to maintain the necessary leadership over their students. Otherwise, they will find themselves being taught by their own men, who, in some details, may be better acquainted with the subject. To the Profession, the value of such a high teaching tone need hardly be pointed out. Furthermore, the student working at some branch of engineering, as he nears the end of his course, can decide for himself whether he prefers that particular branch as his future specialty. He can then begin to supplement his training in the engineering fundamentals by a course of study in his chosen specialty. Such an early decision as to the choice of a life work, if made carefully and discriminately, makes available more time for the attainment of that greater knowledge and understanding of a subject which produces the real specialist. Finally, it starts the student under auspices which will operate for his individual betterment and for the benefit of the Profession.

It may be urged that the grind of the combination course leaves the student no time to attend social functions. In a measure this is true, and hence beneficial, as previously pointed out, but it is not altogether true. The writer's experience and observations lead him to believe that all necessary social functions can be attended without hampering seriously the work at office or school. The course is not one grueling grind, for it is interspersed with a number of holidays and a long summer vacation. By careful and far-sighted planning, a time for almost everything that is reasonable can be found. Of course, numerous social activities, which make up a part of the college life and take up an appreciable part of the student's time, are necessarily curtailed or completely eliminated. The advantages to the student of such comparative freedom from the disturbing and, at times, harassing influences of many social engagements need hardly be pointed out. The impression, however, should not be gathered that the combination-course student is a "grind" simply because he

Mr. Cohen. lives the concentrated life demanded in large part by modern industrial conditions. His lot is not a hard one, and, being always busy, he is in general always happy.

Mr. Swain. GEORGE F. SWAIN, M. AM. SOC. C. E.—The speaker is always very glad to read a paper on education by a practicing engineer, and always derives some good from it. This is true of Mr. McCullough's paper, but, at the same time, there are certain points in it with which he does not agree.

Mr. McCullough states that we must distinguish between engineers and engineering teachers. As Professor Constant has pointed out, the majority of engineering teachers at the present time are or have been engineers. Many of them are practicing and teaching at the same time; and, as Professor Constant states, the younger men who take up teaching are drawn generally from the ranks of practicing engineers. These teachers know probably better than any one else how a curriculum should be drawn up, because they know, not only what the practicing engineer wants, but also what it is practicable for the school to do. It is impossible for a man who has not tried to teach to draw up a curriculum which will work well; he almost always forgets that the problem of engineering education, or of education in general, is not an engineering problem, but a human problem. We talk about the teaching of engineering, but we probably forget what we were, or what the ordinary boy is, at eighteen or nineteen, and we cannot very well theorize unless those things are kept in mind.

One of the most important things to remember is this: Mr. McCullough speaks about the engineer drawing up a specification of what he wants in a man, and the schools filling that specification. The speaker does not think that an engineer can draw up a specification of what he wants, and if he can, the schools cannot fill it, or at least they cannot guarantee to fill it, because they can only teach the student what he can do for himself. The teacher does not give the student knowledge, he shows him how to get it; and if the student does not want to accomplish anything himself, the teacher cannot force him to do it.

The manufacturer, who, for instance, wants to make a spoke of a wheel, can take a piece of wood and fashion it into the proper shape. Now, it may be said that the teacher's raw material is the student, and though the teacher knows what he wants to make of him, he cannot control his raw material; he cannot cut away here and add there, he can simply show the student what he can do for himself. The most important thing in teaching, therefore, is not what shall be taught, but how it shall be taught. That is a truism, a platitude, but it is what we must keep in mind. The important thing is to have the proper atmosphere in the school, in order to make the young men realize that they have great opportunities before them, and that they are being offered a chance to gain physical, mental, and moral qualities which will fit them to meet the problems of life. Mr. Swain.

When the employer of engineers asks for an assistant, he does not care very much what the young man knows; that is of the least importance. He wants a man who is faithful, who is of good character, conscientious, who can think straight, who will not be anxious to stop work as soon as the bell rings, who will be loyal to his employer, who has "gumption," and who can meet emergencies. The amount of knowledge he wants in the young man at the start could be given to him in a very short time. It is the other qualities which are important. The school, therefore, should pay particular attention to the cultivation of the proper atmosphere.

The speaker, of course, has his ideas in regard to what engineering schools should be, and they are very simple. The trouble with the schools is that they try to carry their technical instruction too far; they are narrow; they do not realize that the young man, in starting his career, will not need much knowledge, and if he has the little that is needed, and the other qualities which have been mentioned—the ability to think straight and to take up a new subject and master it—he will be ready for his job, and for promotion, whenever the chance comes. The majority of schools, therefore, should pay more attention to fundamental principles, and not try to carry details quite so far in particular branches. There ought to be a few schools for post-graduate instruction for men who are qualified and can take the time for a more thorough education; and with such an arrangement and the proper kind of instruction, engineering schools should be able to turn out men who will be satisfactory to employers.

The engineering schools are turning out good men to-day, but, like everything else, they can be improved. The schools realize this, and each is trying to remedy its defects as far as possible. One trouble is that parents do not co-operate sufficiently with the schools, the prevailing tendency being to throw everything on the latter. Parents should earnestly co-operate with the school in making the students realize the great opportunities offered them, and the fact that they must work hard; this does not mean to work all the time, but to work hard and endeavor to utilize their time to the best advantage.

Mr. McCullough and one of those who discuss his paper refer to the fact that there are numerous instances in which a man finds himself in after life practicing a different branch of his profession from the one he studied in college, the inference seeming to be that this is a very bad thing. The speaker has never been able to consider it so. The main thing is to follow a line of study in college which will give a man the qualities which he needs to enable him to meet the problems of life. The speaker has had engineering

Mr. Swain. students who subsequently became ministers; others who became lawyers; some who became artists; one or two who have become economists; and others who have gone into business. He has talked to many of these men, and has yet to meet one who has regretted his engineering education. They all admit that such a training gave them what was more valuable than anything else, namely, the ability to concentrate, to work hard, and to get results.

In fact, the speaker has almost come to feel that the study of engineering is about the best training for a young man, no matter what his future career is to be; and if he had a son, whether he was going into business, into the law, or into anything else, he would select such a training for him, because he thinks it would give him, better than any other, those powers which he would desire him to acquire. Besides, he would be dealing with every-day things. Engineering is practical, and engineers are dealing continually with electricity and with mechanics. If these views are correct, we should not be surprised to find many men taking courses in civil engineering and afterward practicing as mechanical or electrical engineers, or *vice versa*. There are very few men who, when they enter college, can feel sure that they are fitted for any specific branch of the Profession. They may know that they like engineering, but their future career is very likely to be determined by some trivial accident. If a man has a good training to start with, and the character and the power that he ought to get at school, he will succeed, and he ought not to be the subject of criticism because he takes up some other branch of work.

With reference to the usefulness of modern languages to the engineer, Mr. Boucher and the author think that modern languages ought not to be required in engineering education. In regard to that the speaker disagrees with them entirely. Recently, he attended the Sixth Congress of the International Association for Testing Materials held in New York City. There were several hundred men at that Congress from all over the world, including the most prominent representatives of that branch of the Profession from almost every country of Europe, one from China, and one from Japan. Almost all those men could speak English; most of them could speak two modern languages. Mr. Henry M. Howe, one of the most distinguished of American engineers, the President of the Association, made his address of welcome in six languages, though the speaker does not suppose that he speaks each of these six languages fluently.

Now, if it is believed that the engineer should occupy a high position among men, not merely that he should be able to do his engineering work properly—building his bridge, laying out his road, or designing his power station—but that he should occupy a high position among men, it appears that a knowledge of such things as modern languages should be encouraged. It is, of course, perfectly true that a man can become just as good an engineer, in a purely technical sense, without knowing anything of modern languages, of economics, or of a great many other things, but a very high standard for the Engineering Profession should be demanded and maintained, not simply in engineering, but among cultured men, and if that is done, a knowledge of at least one modern language, and preferably of two, should be encouraged. Therefore, a student who is graduated and takes a degree from an engineering school should have at least a good reading knowledge of one modern language. The man who cannot get that, can take a special course and get thorough technical instruction, but the colleges and professional men of to-day aim for something broader than mere technical training. Mr. Swain.

William J. Boucher, Assoc. M. Am. Soc. C. E.—This paper is both interesting and timely. Changes have occurred and are occurring in all lines of business, including engineering, and why should not corresponding changes take place in preparation for business and engineering practice. The speaker agrees with the author that schools and professors should aim to fit their graduates more closely for the work to be undertaken immediately after commencement. Very clearly does the speaker remember his first days in engineering work—at the very bottom—and the many very ordinary things he did not know. Mr. Boucher.

The author expresses the belief that engineering schools of the future will require a minimum of six years' work, of which two years will be spent in the preparatory school, but adding two years to the entire time required in preparation for the life work. The speaker believes that such a lengthening of the course would be a mistake. The average age of entering students has increased steadily, due to the increased entrance requirements, until it is now generally about 19 years, which, with a four years' course, makes the graduate 23 years of age; it does seem that this is old enough to start life's practical work, without requiring an additional two years, making him 25 years, or possibly 24 years, if he has been fortunate enough to finish the course in three years. Very few men would be able to do this, for a variety of reasons, chief of which would be the financial one, and those who had their tuition paid by parents or others would hardly feel the stimulus to do it in less than the prescribed time. The speaker was graduated at the age of 21 years from one of the best known mechanical engineering schools, after 14 years of continuous study, and felt and still feels that that was quite late enough to go out into the world. The following advertisement, copied from a recent issue of one of the leading engineering weeklies, appears to emphasize this latter point:

"Position wanted by graduate civil engineer, 25 years, one year's graduate study, open for permanent position in any line of profession, Mr. Boucher. locality immaterial, experience in reinforced concrete construction and sewer design."

Doubtless, that advertisement will be read by several prospective employers who would much prefer that the applicant should have three or six months' practical experience, rather than a year's graduate study.

Mention is made of the fact that colleges admit all who apply and can pass the entrance examinations. This is true, and, as a result, many young men enter engineering courses who are unfitted mentally and temperamentally for that line of work. It seems to be such a waste of good time and effort to instruct young men in technical lines when they would make better mechanics, carpenters, clerks, or farmers. Before applying for admission, a young man should be made familiar, by parents or teachers, with the qualities essential to success in engineering; he should be observed and questioned as to his liking for and ability to solve mathematical problems, and, by various tests, his qualifications should be known to those who would be in a position to advise him in regard to his life work; for, although the engineering and technical studies will not harm him, and in certain ways will prepare him for any work, it would surely be much better for those who do wish to follow engineering as a life work if the classes contained only those and were not overcrowded with many who belong more properly in academic courses and do not care for the engineering training or propose to follow that Profession. This leads very naturally to the observation that so many graduates of engineering courses are found in lines of work in no way related to their training, and it would be largely eliminated if advice and thought were given to the future of the graduate, rather than to the haphazard method, so frequently pursued by parents, of sending their sons to attend an engineering school, because it "seems to be the proper place," or "the proper thing to do."

In a recent address, Alexander C. Humphreys, M. Am. Soc. C. E., President of Stevens Institute, said:

"Many fathers and mothers come to me and tell me that their boys have a natural bent for engineering. Why? Well, they show great aptitude for making electric bell connections, or they are very skillful at the lathe. I generally tell them this: Will your boy apply himself to the hard study, perhaps, to him, the drudgery of mathematics and science? Otherwise, turn your attention to making your boy a good mechanic. The boy must have capacity for mental application besides manual dexterity."

In regard to lengthening the course beyond four years, Dr. Humphreys says in no uncertain language:

"If the course is to be lengthened, who shall determine its duration; if five, six or seven years are needed, then why not seventy, for a genuine student can always learn. One of the disadvantages of a college training, which must be offset by the greater advantages, is that students get to relying too much on their college training." Mr. Boucher.

Further, technical schools are seldom endowed as liberally as the older and better-known universities, and it is a well-known fact that the cost of a student's education is more to the institution than the latter receives in tuition, consequently, the larger the classes the more the institution runs behind in operating expenses, and, for that reason, if for no other, as many students as possible should be deflected into those colleges giving cultural or academic courses. Another very good reason for keeping the classes small, is that, by so doing, the professors come into closer contact with their students, which is always a great advantage to the latter.

On page 1085, the author gives a list, more or less complete, containing his ideas of entrance requirements. This list contains almost the identical subjects required for entrance to Stevens Institute in 1892, in addition to geography (political and physical), United States history, rhetoric, composition, and—probably most important of all—arithmetic. This last, for some obscure reason, the author seems to have overlooked. To the speaker, however, it is a most important subject, one which is constantly used, and in which proficiency and accuracy are most essential, and its use should not be subordinated to the slide-rule or "guessing stick."

As for foreign languages, the speaker is in accord with the author; they should not be required during the course, in spite of the view of one very much respected professor, who held the opinion that the study of foreign languages gives relaxation after the hour of mathematics or physics. A reading knowledge of modern languages is certainly an advantage to the engineer. It should be acquired in the high school, however, and, in order to keep up the practice, reviews of certain foreign technical papers might be required sufficiently often to insure that the student was not losing what he already had. The difficulty in after life is that language studies, probably not any too thoroughly taught in college, are completed (so-called) one or two years before graduation, and, when the latter occurs, the graduate is so "rusty" in his languages that the reading, being anything but easy, is consequently neglected and soon dropped completely; for the busy engineer in practice has all he can do to read a portion—a very small portion—of American technical literature, which each week and month is demanding his attention.

The course in engineering should be made pre-eminently practical. Its use in the future should be kept constantly in view, and

Mr. Boucher. those subjects which will make the fresh graduate useful to his first employer should be elaborated—drafting and drafting-room methods should be insisted on and required. For five seasons the speaker was instructor in a New York City evening school, teaching mechanical drawing. He aimed to make the course useful and practical, devoting only a short time to mere drawing, but advancing the students rapidly to sketching from objects, then drawing the same in a neat and accurate manner, and finally tracing in ink; and, though a season's course lasted only six months, he has the satisfaction of knowing that several of the students, who had never before been in a drafting-room, obtained employment as tracers or junior draftsmen after their one season's course.

The author outlines a course of general engineering study covering four years and designed to produce graduates who shall be well educated on broad lines and acquainted with much that is actually required in their future work. The speaker finds very little to criticize in the work outlined. For several years, Stevens Institute has required, as a part of the course, attendance at lectures and recitations on "business practice," in which attention is given to accounting, depreciation, analysis of cost, specifications, estimates, contracts, and appraisals.

There is probably a diversity of opinion in regard to thesis work, but when properly conducted, and not consuming too much time, some good results may be achieved, for instance, in carrying out a test of a power station at a distance from the college, where the students must rely almost wholly on themselves.

In closing, the speaker desires to mention an incident, which occurred at almost the beginning of his practical experience. Application had been made to a rather prominent consulting and contracting engineer of 1897, who is still in practice, to enter his employ in a minor capacity. The answer, in letter form and preserved as a memento, reads as follows:

"There exists at present no vacancy in my office, but my experience with college graduates has been such that I do not care to repeat that experience."

Fortunately, this attitude is rare, and will become rarer as the products of our colleges and technical schools prove their worth by being immediately useful after graduation.

In conclusion, the speaker desires to make this criticism of all the discussion by professors—that they seem to overlook or ignore the ultimate object of all the teaching, namely, to enable the graduate to secure a position in engineering work promptly after graduation, for that is what 99% of the graduates need. Professors are much inclined to require a too highly finished product, rather than a working knowledge of essentials. Engineers in practice know what they lacked when they started out in the world; they also know of the hours spent on work required in college, which has never been hinted at or needed in practice—work which can properly only come after years of experience in the active practice of the Profession and is only entrusted to those who have obtained standing and reputation by their years of experience; hence it does seem that engineers are very distinctly qualified to have a voice in the making of the curriculum which is planned for the education of their future assistants. Mr. Boucher.

Almon H. Fuller, M. Am. Soc. C. E.—Mr. McCullough has stated that engineering teachers should get together and standardize the courses of instruction. That sounds well, but he seems to have overlooked the fact that each man will have to deal with the situation as he finds it in his respective college, especially in other departments, such as physics, mathematics, and chemistry; and even though they should agree on a standard, there would be difficulty in taking it home and applying it. It is possible that some progress could be made in that way, but the conditions which exist would cause considerable difficulty in effecting a uniform change. Mr. Fuller.

The author also suggests a sequence of the various subjects which differs entirely from that usually followed. By this he hopes to give a certain amount of practical work the first year in subjects which will permit the students to do certain work during the summer, with the thought that if a man stayed by it without coming back to school perhaps the entire Profession would be the gainer. There has been much discussion on the proper sequence of subjects in an engineering curriculum. The usual order is to give much of the so-called cultural work first. Perhaps many would agree that this should be distributed throughout each year.

In talking with some of his own students, the speaker has noticed a greater inclination to take general work in the latter part of the curriculum than in the first. If given in the first part, it is thrust upon them; if available later, many will take it willingly. The speaker has heard practising engineers suggest such an arrangement. Whether or not this is the better plan seems to depend largely on the spirit that can be instilled in the students at various times.

Mr. Green has well said:

"Just what subjects are studied by the one being educated is a secondary matter; the chief concern is that the study shall be inspired and directed in such a way as to develop qualities which further happiness, efficiency, and capacity for social service."

When every instructor recognizes this, and realizes that it includes fundamental training for general resourcefulness—culture if you

Mr. Fuller. please—much progress will have been made. This is of greater importance than the particular arrangement proposed by the author.

Mr. McCullough suggests that a specification for engineering education be written by engineers. Professor Swain thinks that would not be practicable. Perhaps it would not be. The speaker can see many objections to it. However, as an engineering teacher, he would like to see the specification. He would welcome the opportunity of examining it, of comparing it with the present curricula, and of attempting to adapt it to the conditions that exist in the institution with which he is connected. If a representative committee of engineers would take the trouble to write such a specification they would deserve the thanks and possibly receive the approbation of the teachers. As Professor Swain has said, unless the men who write it were very closely in touch with the engineering colleges, it might not be very useful, but it seems to be entirely possible that it might bring out many points which engineering instructors could adopt with splendid advantage.

Office atmosphere may well be kept in mind in conducting courses in drawing and design. At the same time, it will not do to lose sight of the fact that, in the office, the intent is to mould the entire force into a smoothly working machine which will produce the greatest output; while, in college, the purpose is the development of the individual.

Mr. Allen. Walter Hinds Allen, M. Am. Soc. C. E.—In the first part of the Nineteenth Century the young man who desired to become a lawyer secured his professional training by going into some law office where he would read law for several years. Later, law schools were founded, and, by attending one of these, a much better legal education was possible. These methods, however, did not afford a broad education, and, nowadays, the majority of law students first acquire a general college education, waiting to get their technical education until the age of twenty-two or later, when the mind of the young man is so much better able to comprehend and master the more intricate technical problems. Some of the modern law schools will admit only students who have received a Bachelor of Arts degree or the equivalent. These schools recognize the fact that general education is essential in order to produce the best lawyers and citizens; and that the man of twenty is not able, in most cases, to get the full benefit of his professional study.

This same condition is true to some extent in the study of medicine. Of course, there are and always must be schools of law and of medicine which admit students whose education has not advanced beyond the high school. Not all young men are able to afford the time or money necessary for a college course, and it would be most unjust to deprive them of an opportunity of entering these professions. It is generally recognized, however, that such a course is a desirable preparation for professional study. Mr. Allen.

At present the engineering schools of the country are at that former stage of the law and medical schools, when a previous college education was not a requisite for admission. The Engineering Profession is behind its sister professions in this respect, for a good general education is just as essential a preparation for engineering study and to produce the best engineers as for any other profession. Such general education need not be exactly the same for all professions. For one who intends to study engineering, much preliminary scientific study may be undertaken in mathematics, physics, and chemistry; but history, economics, literature, modern languages, and rhetoric should receive considerable attention. These subjects will prove of value, not only to the engineer in practice, and particularly as he attains more prominence in his profession, but they add to his culture and ability to stand well among his fellow men. They increase his power of enjoying the higher things of life.

An undergraduate college course is completed ordinarily at the age of twenty-two, at which time the young student, having reached the more serious period of life, is ready to take up the technical preparation for his life work. If he has finished his studies in pure mathematics and other elementary subjects, he can get a thorough engineering education with two or three additional years of study.

In another respect, the engineer may well profit by the example of the lawyer or doctor. After graduation it is a very common thing for these men to enter law offices or hospitals and work for one or two years with little or no compensation. They do not so much consider the financial side as the opportunity afforded to observe the best practice and to supplement their study at the professional schools. In the speaker's opinion, it is entirely wrong to assume the attitude that the man who has just completed his technical school course should begin immediately to earn good pay. He is not yet of any great value in his profession; the man who has not had the opportunity for education, but has started his practice at an early age, is, for a number of years, of much greater value to his employer. The trained man, however, has far greater possibilities in him, and nine times out of ten becomes the better engineer after he has had some years of practical experience. He himself should realize this and be content in his first years to make monetary compensation a consideration secondary to securing the best experience.

The speaker had occasion last winter to investigate the opportunities offered for certain young men, technically trained, and graduates of an engineering school. who had had two years of practical experience, to take a course of study that would give them a broad civil engineering education. These men were about twenty-five years of

Mr. Allen. age, good students and well equipped in mathematics and some branches of civil, mechanical, and electrical engineering. As far as the investigation disclosed, there is only one Eastern university or engineering school which has a regularly organized graduate school of engineering. This has been started recently, and marks, in the speaker's opinion, an epoch in engineering education in the United States. The number of young men who take up the study of engineering in a graduate course, after obtaining a general college education, is steadily increasing, and the opening of this graduate school is an index of the trend of engineering education.

It is a good omen, too, that this and other engineering societies are taking interest in the education of those who later will become engineers. The members of the Profession by their advice and interest can exercise a strong influence in securing the best training for their successors. This cannot be done effectively by bringing pressure on the schools themselves and by trying to dictate what they shall teach. The schools will furnish that kind of education for which there is a strong demand from the students themselves.

Outside engineers can do far more good by using their influence with young men who are intending to become engineers, by inducing them to secure a good general education first, and to pursue their technical studies afterward. The practicing engineer should encourage the beginner to take a broad view of his profession, to look to the future, and to map out his early training and practice with a view not so much to immediate financial success as to attaining ultimately the top of his profession.

Mr. Stengel. C. H. Stengel, Assoc. M. Am. Soc. C. E.—In order to substantiate some of the facts brought out by Professor Swain, pertaining to the statement that engineers should be graduated at the age of twenty-one in preference to a more advanced age, to give them an early start in the Profession, the speaker would state that he has had in his service a number of young graduate engineers, and, after careful observation, has found that their intellects are at a more advanced stage of development, their work more accurate, and themselves better men on the average, at the ages of from twenty-three to twenty-five than at twenty-one. The more mature the mind of the student at the time he is laying the foundation of his career, the greater are his intellectual powers, principally in absorbing and retaining the knowledge he is gaining, to develop his logic and reasoning.

When the young man enters college intending to take up Engineering, his course should consist in mastering thoroughly and conscientiously the fundamental principles which form the basis of the Profession in all its branches; then, with his power of application, he should be able to fit himself for any of its branches, and his rise will soon be assured, if his energies, resourcefulness, and ambition are applied to his work. Mr. Stengel.

As stated, it is the personality and self-reliance of a young man entering the engineering world, together with the thoroughness in which his mind is developed in not only the fundamental principles underlying his Profession, but in careful analysis and accuracy in the performance of any work he may pursue, that mean success; and to accomplish this he should have the full confidence of his tutors and the co-operation of his parents (as stated by Professor Swain) in the moulding of his career.

Charles Warren Hunt, M. Am. Soc. C. E.—The general subject of the education of the engineer is of great interest to the speaker, inasmuch as, for more than twenty years, he has been in a position which has enabled him to form an opinion of the results of modern technical training. Mr. Hunt.

Professor Swain has stated certain logical, broad, and proper basic principles on which engineering education should be founded, nevertheless, in the speaker's opinion, the tendency of the modern technical school is to become more and more narrow.

A boy who wishes to become an engineer must decide, practically upon matriculation, which special branch of this great Profession he will follow: Civil, Mechanical, Electrical, or some other. During the course in whichever specialty he chooses, he is forced to spend many hours in working out details of that specialty (in many cases without even a suggestion of a study of modern languages, history, literature, or in fact of any of the humanities), and, after four years of hard grinding, is graduated as the particular type of engineer indicated by the title of the course pursued. He must then secure a position for which that preparation is supposed to have fitted him—he has no other option—and then follows a period of years during which, in the struggle for existence, his nose is kept so close to the grindstone that he has no time even to look about him for broadening influences; so that, when he reaches the age at which he should be most productive and efficient, he is not fitted to take and keep the position, in the social, political, or business life of the community in which he lives, to which his intellectual attainments and constructive skill entitle him.

It is trite, but true, to say that the engineer is the pioneer of all civilization, as well as one of the most important factors in its advancement; and it is then most natural to inquire why his position among his fellows is not commensurate with his achievements. In the speaker's opinion, it is because he is not enough of an all-around man; he is not broad, not capable of thinking clearly and quickly along any other lines

Mr. Hunt. than those to which he has given up all his formative years. He does not, therefore, succeed in impressing his personality on his fellow-man, although he has not the slightest difficulty in so doing on his fellow engineer.

The speaker believes that the modern system of engineering education is, speaking broadly, responsible for this condition. He does not know enough to attempt to discuss any of the details of curricula or class-room, but would like to go on record as believing that the specification for a properly equipped technical graduate should not be that he should be able immediately on leaving school to be valuable to an employer in any specialty, but that first of all he should be full to repletion with knowledge of the fundamental laws and principles of the exact sciences on which the sound practice of engineering in all its branches must be based; and, in addition to this, his attainments outside of technical matters should be broad enough and fundamental enough to enable him to become a man of the world. It is time enough for him to specialize when he has found out what he is best fitted for, and what his opportunities are. To be successful, an engineer must not only be able to do the technical work which comes his way, but he must be able to get it, and his ability to hold his own with men of other professions and in the world of business must ultimately decide whether he shall be in fact, as well as in name, a professional leader in the community, or continue to be regarded by the general public as a sort of an upper class mechanic.

Mr. Blanchard. ARTHUR H. BLANCHARD, M. AM. SOC. C. E.—It is not the speaker's intention to discuss Mr. McCullough's paper from all standpoints, but to call attention briefly to certain phases of the subject which might not be treated in the general discussion.

The speaker wishes to emphasize the author's recommendation that advanced specialized work can be taken profitably by graduate engineers, provided the period of attendance and other details are arranged satisfactorily. Up to this date, very few examples of educational work conducted along these lines are at hand. One case, however, which is conducted on the plan proposed, is that of the graduate courses in highway engineering at Columbia University. The period in which these courses are offered is from December 1st to April 1st. Hence an engineer desiring to take all the graduate courses in highway engineering and allied subjects, which fulfill the requirements for the Master's degree, will necessarily be in attendance for two winter periods, the equivalent of one collegiate year. Although candidacy for the Master's degree requires as a prerequisite a Bachelor's degree, nevertheless, mature men are admitted to any courses for which they are qualified, and may take any number of courses.

Mr. Blanchard.

As this plan is somewhat of an innovation in engineering education, it may be of interest to cite certain facts in connection with the attendance during the winter period of 1911-12, which was the first period under this plan. Although the graduate courses were not brought to the attention of engineers until November, 1911, there were in attendance fifteen men affiliated with highway work, thirteen of whom registered as candidates for the Master's Degree. It is of interest to note that this group included men connected with State highway departments, contractors' organizations, municipal departments, engineering-sales departments of manufacturing companies, county highway departments, and consulting engineers' offices. The experience of these men ranged from one to twelve years. They came from widely distributed localities, Massachusetts, New York, Pennsylvania, Maryland, North Carolina, Alabama, Panama, and British Columbia being represented.

The idea, as suggested by Mr. McCullough, that men taking advanced courses should work on special problems is followed out at Columbia, and it is of interest to note that the founding of several research fellowships by various manufacturing companies is under consideration. The research workers holding these fellowships will investigate problems of particular interest and value to the manufacturing concerns founding them. It is expected that many problems of wide interest to those engaged in highway work will be thoroughly investigated through this medium.

The speaker hopes that the author will elucidate his remarks relative to the injection of an office atmosphere into the classroom. Does the following plan, adopted in connection with the graduate courses in highway engineering at Columbia, approach Mr. McCullough's ideal? This plan consists in the employment of a large number of experts in various fields connected with highway work to act as non-resident lecturers in highway engineering. These lecturers cover certain subjects with which they are particularly familiar and their topics form an integral part of the various courses. Although the regular officers of instruction are actively connected with highway work or allied subjects, it was thought that lectures, based on the plan outlined, would tend to broaden the viewpoint of the graduate students, besides bringing them in contact with men of the highest standing in this branch of the Profession.

Mr. McCullough evidently does not fully appreciate the value of a training in French and German. He considers this subject from two standpoints: first, ability to converse in a foreign language; and, second, ability to read foreign literature. The speaker thoroughly agrees with the author in his implied criticism of the time wasted, both in preparatory and technical schools, in the attempt to acquire

Mr. Blanchard. the ability to converse in French and German. He feels, however, that an entirely wrong impression is given when it is intimated that, for those who have never taken French or German, only a few weeks' work is necessary with a phonograph or in special schools in order to acquire ability to transact business or discuss engineering problems with those speaking a foreign language. Based on the speaker's experience with the use of foreign languages in Europe, and his knowledge of the methods used in teaching French and German in preparatory and technical schools, the following recommendation is offered for consideration: In all foreign language courses for engineers the entire time should be devoted to a thorough study of grammar and to translations. The time now devoted to the reading of French and German in the original is generally wasted. In many cases the pronunciation used by American teachers is poor, and hence those who attempt later to converse in foreign languages must forget the faulty pronunciation acquired previously. An engineer who is called on to use French or German in Europe will find it profitable, after mastering the vocabulary covering his particular field of work, to devote the requisite time to association with a French or German teacher and to living with a family where only the foreign language is used, in order to acquire the native pronunciation and have an opportunity to converse in the foreign language.

The author uses the common argument that "everything of value appearing in the foreign papers is quickly translated." Naturally, the deduction is that engineering literature of value to American engineers is translated and reprinted as it appears in the foreign press. In the field of highway engineering, such is certainly not the case. Before devoting a year to the investigation of the construction and maintenance of roads and pavements in foreign countries, the speaker attempted to review thoroughly the practice of the leading countries of Europe. It was found, however, that the so-called translations referred to gave a very inadequate idea of current practice in foreign countries. The result of the speaker's investigations showed that European engineers had adopted many methods, in connection with the construction and maintenance of highways, with which American engineers were not familiar, and likewise that the few references to this practice in the English press gave a perverted view of foreign practice. That American engineers in many fields may profit materially by thorough study of foreign practice does not require extended argument. Many instances in highway engineering have occurred in which both failures and successes of foreign engineers have been duplicated as experimental work in the United States where such work would not have been undertaken if the experimenters had been familiar with the results of foreign practice. The speaker has in mind an experiment described by an American engineer, and labeled as a new invention, which had been in use for a number of years in Great Britain, Germany, Austria, and France, and had been described in foreign periodicals. The practice in highway engineering in English speaking countries is very well covered by the technical press of the United States, Canada, and England, but it is the exception to find the best articles printed in the *Annales des Ponts et Chaussées, Annales des Chemins Vicinaux,* and *Le Génie Civil,* of France; the *Annales des Travaux Publics de Belgique;* the *Zeitschrift für Transportwesen und Strassenbau,* and *Der Strassenbau,* of Germany, translated and reprinted or abstracted in the technical press of America. Mr. Blanchard.

Philip W. Henry, M. Am. Soc. C. E.—More or less has been said about education in different branches of engineering, as if it made considerable difference in a man's career whether he takes a course in mechanical, mining, electrical, or civil engineering. It is difficult to differentiate these courses, and the speaker does not think it is necessary to do so. It is the quality of instruction that counts, rather than the subject. A course in mining engineering, properly given, will better fit a man to be a mechanical engineer, than a course in mechanical engineering improperly given. The degree which a man obtains on Commencement Day does not make him an engineer, but indicates, or should indicate, that he knows how to work intelligently on any engineering problem which is set before him. In the class-room he has been compelled, every day of his four years' course, to concentrate his attention on a definite problem, and demonstrate its solution on the blackboard or in some other concrete way. When, after graduation, he takes a position, no matter how humble or in what branch of engineering, he still finds that there is a daily problem to solve, and that, through his training in proper methods of application, he is able to solve it more easily, and thus advance more rapidly than a man, who, with the same mental endowments, has not had the advantage of the same kind of training. In addition to this mental training, good for any kind of business—dry goods or otherwise—the graduate engineer has the advantage of knowing where to go for any detailed technical information bearing on the subject in hand. Mr. Henry.

Many students in engineering schools have only sufficient means to carry them through the course, and, of necessity, must accept the first position open to them. If, therefore, a man who has taken the course of mechanical engineering finds that the only opening is in the office of an engineer whose specialty is sewer construction, he should not despair, but should take that or any other position which may offer advancement, feeling confident that his training will come into use and that he will have the advantage over all

Mr. Henry. his competitors in his ability to work thoroughly and intelligently. By steady application and by taking an interest in his daily task, he will find advancement sure, even though it may not be in that branch of engineering for which he originally prepared himself.

Mr. Rogge. JOHN C. L. ROGGE, M. AM. SOC. C. E.—Professor Swain has stated that when one is studying engineering, he cannot tell what business he will follow ultimately. The speaker would like to say a word or two in reference to engineers engaged in lines of business other than engineering, and to show how circumstances alter cases, using his own career as an example.

He was educated as an engineer and followed the Profession for about twelve years. During part of this time he was employed in one of the New York City Departments where he rose to be Chief Engineer. While thus employed, he was so impressed with the success of various contractors who worked under his supervision and who had little or no education, that when the opportunity came, he resigned his position and entered the business world. The venture was a success, and he has never regretted the change.

While a man's environment, opportunity, and temperament are always large factors in his success, the speaker believes that an engineering education would not be found to be a handicap in any business or profession, because it trains one to reason, to plan, to be keen in observing, to be able to make quick and accurate decisions, and not to take anything for granted, all of which are valuable to one who is in commercial life. A prominent New York lawyer, who was graduated from Stevens Institute as a mechanical engineer and subsequently took up law as a profession, informed the speaker recently that his engineering education had been of great benefit to him in the study of law.

A man who has followed the Engineering Profession for a considerable length of time, however, is apt to be timid as compared with the every-day business man, because of the extreme accuracy demanded by engineering work; but if he will follow engineering just long enough to learn to apply what he has studied in practice, he will then be ready to take up any other line of work or business which may suit him better, or in which there are more financial returns.

To young men studying engineering the speaker would say that there are many opportunities in the commercial world where an engineering education can be used with profit.

Mr. Higgins. CHARLES H. HIGGINS, M. AM. SOC. C. E. (by letter).—This paper is very interesting, expressing as it does, a natural and not uncommon point of view toward this vitally important subject.

The author appears to take for his premises the following: "The engineer should merely give to the teacher his specifications for a good assistant, and the teacher should try to follow the specifications." For those who accept the foregoing, it can only be a matter of deep regret that the author did not furnish a sample copy of the specifications, including a form of contract and a notice to bidders. The brief description contained in the paper, can, in no wise, alleviate the disappointment felt in not finding the proposed specifications for the finished product. Mr. Higgins.

The author states that "it should not be a difficult matter for teachers to standardize a course of instruction in engineering"; but is it not a little too much to expect of those "whose sole function in life is to prepare assistants for the engineer, and train those who in the future will be engineers," before they receive copies of "a specification for a good assistant" and know the conditions to be imposed by the contract? To illustrate: Some forms of contract contain a clause providing for liquidated damages to the amount of $100 per day for failure to complete the work within the specified time, in full accordance with the specifications. The contracting teacher would have to take such a clause into account in preparing his bid and planning his future course. In all fairness, a copy of the specifications should be sent before the method of carrying on the contract is required.

Discipline and specialization, of course, are good, but is it not a little severe to prescribe, even for teachers, a "sole function in life"? The writer would not be quite so severe; he thinks that he would allow the exercising of at least one more function, even in the case of a hardened offender.

Many engineers not only receive their assistants from colleges, but they send their sons to them, and that gives another point of view.

There is much in the latter part of the paper which the writer would like to endorse heartily, particularly the advantage to be gained in arranging the course so that a man will have obtained some training that will serve to recommend him for a position in engineering work during the summer vacation following the freshman year. Also, the recognition, in the reference to 6 universities and 200 technical schools, of the fact that there may be a distinction; and, above all, the emphasis laid on the importance of a training in English, including public speaking, and in economics.

Perhaps engineers expect too much as assistants, of young men just out of college. Professors of engineering probably know the difficulties of training in college, just as practising engineers do of continuing that training later in the office. Should the student be trained in details as suggested, it may very well be that he will not detail any steelwork for several years after leaving college; meanwhile, methods of detailing will have changed, or he will find that the office he enters has methods which he must learn to follow.

Mr. Higgins.

Is it the function of the college to take the place of office and field training? The writer thinks not. What it can do is to educate its students in the underlying principles of Nature, and broadly, in the methods of their application, for the use and convenience of Man; and make him more receptive to experiences and capable of interpreting them in the light of the known laws of Nature.

After all, there are distinctions between skill, knowledge, and education. The training which makes the best assistant during the first year out of college is not by any means of necessity the best for the recipient. The college may owe something to the practising engineer, but it certainly owes vastly more to the students and their parents. The human element will always remain. After all is said and done, engineering is for men and not men for engineering.

Mr. Buerger.

Charles B. Buerger, Assoc. M. Am. Soc. C. E. (by letter).—Mr. Green has stated that the aim of immediate usefulness to the future employer may properly be made a secondary consideration in the determination of the curriculum; and it is quite likely that this aim of early usefulness would fail. A course, or a student's electives, may be intended to fit him for a particular position, such as assistant to a consulting engineer, and such position he may never have occasion to fill. Outside of domestic servants, the employé in a subordinate capacity is far from being a free agent, with "liberty" to contract, the Court of Appeals notwithstanding, his occupation being rather a matter of accident than of his wishes or qualifications.

The best curriculum is the broadest one; one which of itself will fit the student for no special position, but will give him the capacity to learn most readily the duties of any one of many possible positions; and his practical education will be obtained, as Mr. Green points out, after he has left school.

Mr. McCullough has not dwelt on the method of teaching, and that is a feature which a teacher should be best qualified to decide; but any one who has been a student has a right to a small voice in the matter. As a rule, the teaching system now comprises 8 months of study per year, 20 hours per week, the time being divided approximately between lectures, quizzes, and the laboratory, the last including shop, field, experimental, testing, and drafting work. In addition, students are expected to put in from 4 to 5 hours each day in private study. The writer would substitute a school year of 50 weeks, with 44 hours of study per week, say 8 hours each for 5 days, and 4 hours on Saturday. He would abolish all lectures and all quizzes, leaving only the laboratory work and the examinations of the present system.

Of the college men with whom the writer came in contact during their student days, numbering, perhaps, 500, four-fifths went through the prescribed courses in a perfunctory way, regarding them as necessary evils, the solace being the shortness of the school hours, and the

Mr. Buerger.

time available for other things. Friends of these students in the commercial world were spoken of as being at work; the students themselves were at college, never at work in college. These are only words, but they represent correctly the student's point of view.

The boy of 16 who goes into the business world puts in 8 hours at his daily task, and be he clerk, mill hand, or rivet boy, he takes this length of time as a matter of course. It does not occur to his employer that the boy should work 4 hours a day at his shop, or office, and do the additional 4 hours' work at his own home, should it be work that could be done at home. If the employer did this, he would get exactly as much done in the 4 hours at home as the college student does in his home study. Nor could this boy get 4 months' vacation a year, even without pay, for the employer would consider steadiness of application a primary qualification.

This lengthening of school hours and elimination of home study has not the same meaning as the recent changes in the New York public school system, which have eliminated in effect any study of any kind on the part of the pupil. It is, in fact, the reverse. The study time is moved into the school hours, and these school hours are doubled thereby. The study time is made an essential part of the course; it is even made the only essential, and replaces entirely all lectures and quizzes which, at present, occupy the greater part of the school hours.

This teaching method, then, consists of, say, 32 hours of study per week under supervision, and 12 hours of the various courses belonging under what has been called laboratory work.

The ordinary school lecture is an abomination. In the Stone Age, it was no doubt a proper means of teaching; now there is no excuse for it. It is true that many instructors cannot find books which they consider suitable. With their judgment, the writer will not quarrel; but, even then, they question the value of their lectures by giving the students the substance in mimeographed sheets.

The ordinary quiz is a useful means of teaching the instructor what the student knows, but it is no help in teaching the student what he does not know, and that is what he is after, always granted that there are some capable teachers who make a success of these methods.

Studying under supervision means necessarily individual instruction. This would mean a larger number of instructors, except that it is entirely feasible to use the more efficient students as aids to the instructor to assist the less efficient ones. It would be better, also, to change the terms to correspond to the change in method, and say that the student instructs himself from his printed matter and that he has a supervisor to render necessary aid.

The writer thinks that further elaboration is unnecessary; it can be expressed in two sentences:

Mr. Buerger.

1.—Make the student put in a full day's work every day, and watch him so that he does it.

2.—Apply correspondence-school methods to the college, with the additional advantage of personal contact and personal help.

Such a system will make the student, not a passive receiver, but an active studier, and when he is that, there will be little complaint as to his curriculum.

Mr. McCullough.

ERNEST MCCULLOUGH, M. AM. SOC. C. E. (by letter).—As a teacher, Mr. Garver feels that the writer has presented a paper criticizing teachers, whereas the intention was to assist those in engineering schools by giving suggestions for the better preparation of embryo engineers. The attitude of mind often warps judgment, and, as Mr. Garver read things into the paper that were not there, the writer would suggest that he read it again. For his information, it may be stated that in the Michigan Mining College, Houghton, Mich., the University of Chicago, Chicago, Ill., and Valparaiso University, Valparaiso, Ind., the system of 12-week terms, with new classes in every subject beginning with each term, has been in use for many years. The writer fails to see that these schools have a larger proportion of teachers to students than other schools. The professors have to work a little harder than the majority of professors, almost as hard, in fact, as the majority of engineers in active practice, when the latter are fortunate enough to have a job. The writer understands that a number of private schools also have their doors open throughout the year, and the proportion of teachers to pupils is about the average.

Captain Pillsbury is a graduate of, and has been a teacher in, the finest vocational school in the world. The students are selected after a very careful and severe physical examination followed by a no less severe mental examination. Their conduct is rigidly guided throughout four years of as strenuous work as men can do and survive. This training, however, is in preparation for a position guaranteed to all graduates. A man is even paid while learning. A few years after he has reached his prime, and long before he has outlived his usefulness, he is retired on a pension which, to many engineers in private life, looks like affluence. Criticism made by a man trained under such a system is not as valuable as it might be, for he knows nothing of the trials and tribulations of the average engineer, so long and humorously referred to as a "job chaser." The average student of technical schools has to go through school on very short allowance, and many have to earn the money. On his graduation, no kind Government engages his services. He must strive hard to get a position, and must compete with men having less schooling and more practical experience. The competition is becoming more keen each year. The following* illustrates this point:

* Extract from an article by Edgar Marburg, M. Am. Soc. C. E., entitled, "Engineering Graduates and the World," *Engineering News*, July 4th, 1912.

"It may be of interest to add, that of the total number of graduates, 1 258, beginning with the class of 1873, more than one-half have graduated since 1904." Mr. McCullough.

The graduates referred to are from the Engineering Department of the University of Pennsylvania. The writer has obtained printed matter from other schools, and a study of the subject shows that the foregoing fact is true of the majority of engineering schools. There is no reason for such an increase except widespread advertising, and, in the paper, an endeavor was made to point out a way of altering the present sequence of studies, in order that there might be a continuous elimination of the unfit, beginning with the first year in school. The writer is sorry he failed to make his meaning clear.

The writer also fails to understand where his critics gain the impression that he advocates less mathematics than the present curricula provide. He said "Either mathematics should be taught in a manner that will provide the student with a useful tool, or the time should be given to some other subject." He did not decry the value of a rigorous course in pure mathematics, but he did criticize the slipshod manner in which the subject is taught in too many schools. However, as the question has been raised, it may be said that many eminent educators have stated lately that too much emphasis has been laid on the value of mathematics as a cultural study. That study develops only the mathematical portion of the brain. It does not tend to broaden the mind, and therefore, should be taught rigorously only to those persons who may be apt to require it in later life. It is more difficult to remember than language, and for those who have no mathematical bent it is time wasted to teach anything more than high school mathematics, purely for cultural purposes. The writer fails to see why a "practical" course cannot be "rigorous," and would recommend to his critic a perusal of the book referred to in the paper.

Mr. Constant's discussion meets with the writer's approval. He has evidently read the paper carefully, and it is thought that he must have been in far better touch with actual conditions than the majority of teachers in engineering schools. He goes to the heart of the matter in the following paragraph:

"After all, however, it is not so much the precise nature of the curriculum as the manner in which the subjects and the students are handled that is important. How to bring out the very best in every man, to stimulate his interest and devotion to his work, and, at the same time, to eliminate the lifeless and the small group of deficients always to be found at the lower limit, who, by sheer persistence, in point of time, finally get through, no more fit, perhaps, at the end than at the beginning—this is the real problem of the engineering school."

Compare the foregoing with the last three lines of the second paragraph of the paper.

Mr. McCullough.

For many years the writer tried to get into teaching work, but for three reasons was unable to do so: First, he had never been a teacher in a school of university grade; second, he might have received minor appointments carrying considerably less pay than he could average in practice; third, he was voted against by the faculty in four institutions because the professors said their experience with teachers having many years of practical experience was as a rule unhappy. The man of more than ten years' practical experience does not mix well with the average faculty man. The result is an emulsion rather than a mixture.

Consequently, the writer has been compelled to satisfy his desire to teach, in part, by conducting classes in vocational subjects in institutions to be found in most large cities.

Few teachers in engineering schools are there from deliberate choice. Too many have entered the work because a teaching position was open at a time when they were out of a job. They took the low pay of an instructor to tide them over a winter, and ended by staying permanently. A large part of a teacher's work consists of lecturing, and few men are harder to listen to than the average teacher in an engineering school. A friend once said of a widely advertised professor, "I never listened to a man so reluctant to part with his conversation." The students who had to sit in his classes said of him that he lacked tact, and was so difficult to follow that they failed to see why he was kept year after year. The writer believes there is a far larger proportion of unfit teachers than of unfit men in any industry. Is it any wonder that a man like Mr. Taylor should prepare a paper entitled "Why Manufacturers Dislike College Graduates?" The writer thanks Mr. Constant for his conscientious discussion.

Mr. Green's discussion reads like a high school thesis, and does not contain a single original thought. All he wrote has been written before, and the writer has read such things in discussions on engineering education printed two or more generations ago. This is not a new discussion, by any means, neither can any one put forth really original ideas on the subject. He can only voice the ideas of groups he voluntarily seeks to represent, to the end that there may be improvement. "Qualities make up education, not knowledge." How often that idea is expressed in different words. Lately, some big business man said "I find it is not so much what a man knows, as how he knows it, and character coupled with opportunity, rather than knowledge, determines success and failure." Life is one-half opportunity, one-third ability, and one-sixth technical knowledge as Mr. Green and other young men graduated as engineers will discover sooner or later. It is easily possible to give too much scientific and technical instruction to some young men who would have been served if sent out earlier with somewhat less education, as education is defined in the usual academic sense. Mr. Green insists on the duty of the employer to educate the engineers he employs. Does he not know that this is precisely what every employer does; and it is also very costly education. The ultimate consumer pays for it. The writer insists, as the result of twenty-five years' experience since leaving school, that the main object of the majority of engineering schools is to train young men to be competent assistants, and, if blessed by opportunity and backed by ability, they may develop into engineers. First, we must define an engineer, and an attempt to do this was made in the opening paragraph of the paper. Mr. McCullough.

The writer has interviewed every man whom he found willing to talk, and in this way has obtained the opinions and ideas of many hundreds. The majority took up engineering because they wanted a college education, and their parents were willing to give it to them provided they studied engineering, which popularly is supposed to be very lucrative. The prevailing opinion is shown by the effect the Panama Canal had on the enrollment in engineering schools in 1900, many people trying to have their boys graduated in time to secure a position on that work when it would start, in 1903 or 1904. The writer has been told by forty-seven young engineers who were graduated about that time that this was their sole reason for studying engineering. Contractors and other employers do not take engineers fresh from the schools; they take minor assistants. In fact, the fresh graduates usually have a hard time securing employment, few men caring to give them the necessary experience. They must take clerical work, or anything they can get, and then depend on their native ability to go up. They are, in effect, educated by the employer; not as the bricklayer is trained, because there are few bricklaying schools. When trade schools become as relatively plentiful as engineering schools, the large employers will discontinue whatever instructional courses they are now presumed to have, although, in his knowledge of such courses, it is admitted that Mr. Green seems to possess more information than the writer. The writer asks Mr. Green to read carefully the title of the paper and the third page. Engineering education was not therein dealt with as a training in pure or applied science. The title is "Engineering Education in its Relation to Training for Engineering Work," therefore, education was discussed purely from the vocational standpoint.

Mr. Saurbrey, in his opening paragraph, takes occasion to mention the difference between "Engineering Education" and "Engineering Training." A teacher of business once said "When writing a telegram, use no punctuation marks. Hand it to a stranger to read, and if he gets your meaning then send it. If he does not get your meaning, re-write it; but remember, no punctuation." One often neglects to write so clearly that he can be free from criticism by men

Mr. McCullough.

who split hairs and hold rigorously to definitions. Mr. Saurbrey objects to the use of the word "pose." As he understands that word, the writer was unfortunate in using it, and perhaps might have said: "The well-read man is generally able to pass as a cultured man." To some, the use of the word "pass," in this connection, still bears too strong a resemblance to a game of poker, therefore is again "pose." Mr. Saurbrey has dilated too much on the unfortunate selection of that word. The writer meant to say that the man who reads deliberately from choice, instead of having manufactured learning stuffed into him by teachers, generally makes the best impression on people who look on the possession of real knowledge as being an evidence of culture. The "poser" was the last man in his mind when he penned the criticized sentence. Mr. Saurbrey goes afield, however, in leaving the technical school and going back to the home and the common schools. The writer insists that the technical and engineering schools take the raw material as it is delivered, and, from the first day of school, begin to put in motion a proper law of selection; that and nothing more. His curriculum is practically that of all technical schools of to-day. His arrangement, however, departs from the common one for the purpose of assisting in the early elimination of the unfit, and the dilation of the sense of perception on the part of those who took up the work ignorantly and have in them the germs of engineering ability. A liberal offering of electives gives every man full opportunity to travel as far as he likes in the paths of the scholar, nay, even in the path of the dilettante in matters bookish. Those who like more mathematics than is required can indulge their taste. Those who hanker for the ability to read foreign languages can have their hankerings satisfied.

Mr. Cohen has made a real contribution to the discussion, and is pretty well in accord with the writer in his ideas on the subject, as specifically dealt with according to the title of the paper. Mr. Stengel seemingly has some difficulty in getting at fundamentals. The writer believes that, when a young man is shown how to do a thing and then, in the course of his studies, is given the reason, he is far more likely to take an interest in his work than if he is given a two years' dose of "why" before getting at the "how." The writer, in handling his classes, obtains the best results by training men in doing things, and then giving the reasons when some curiosity is excited. Take the planimeter for example: It was required by a higher instructor that the pupils give the mathematical theory of the planimeter in an examination. The writer first taught the use of the planimeter, and areas were found by it. Then he bent a wire before the class and made a hatchet planimeter. With this crude instrument areas were measured with an accuracy that was surprising. After this preliminary treatment, the elucidation of the theory and the presentation of the fundamental equations involved no work, but was attacked with zest. However, to this day, the writer cannot see what difference it made, for the instrument is a commercial product and no engineer is going to make one, unless it be the hatchet planimeter in its crudest form; and then he does not have to know the theory. Mr. McCullough.

The writer is pleased to learn of the work being done at Columbia University, as described by Mr. Blanchard. The injection of an office atmosphere in graduate courses is well attended to by the method adopted by Mr. Blanchard when it is considered that every man taking the course has had undergraduate instruction, and, subsequently, considerable practical experience. Such men, however, do not require the office atmosphere, because they understand the conditions of engineering life. They really are after the academic side. The office atmosphere mentioned by the writer is something which the undergraduate should breathe from the first, in an engineering school. It cannot be imparted properly when "inbreeding" is the rule in selecting members of the faculty. No man should be employed as an instructor in an engineering school until he has had not less than five years' practical experience of a good character. No graduate of the school should be appointed an instructor, for there are plenty of engineering schools turning out fit men. A man should not be an assistant professor until he has served some time as an instructor; and a graduate of the school can be appointed as an assistant professor, provided he has had not less than five years' practical experience and has also served some years as an instructor in some other school.

Willingness to accept a teaching position should not count so much as a proven ability to teach. An engineering teacher should be a fluent and not a hesitating talker, as so many are. He should be interested in his work and in his students. The writer knows some professors who have nothing to do with their students outside the classroom, and these professors are not men of high standing, it being his observation that the higher standing the teacher has as a man the more of a common man he is with his students. Given teachers with practical experience who know the ups and downs of the "job chaser," the proper tinge of office and works atmosphere can properly be left to them. The writer knows what he would do had he the opportunity to conduct an engineering school, but cannot go into details in a paper such as he presented nor in any discussion. If the teacher cannot eliminate a proper amount of academic atmosphere and substitute a wholesome amount of office and works atmosphere, then he belongs in the liberal arts department rather than the engineering department of the school in which he holds a position on the teaching staff.

Mr. Blanchard does not fully understand the writer in his remarks on the teaching of languages. His criticism of language teaching was

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similar to his criticism of the teaching of mathematics in the average school. The writer had the usual high school Latin and Greek. He also studied German years ago and later French. Some years after leaving school he obtained a position where a knowledge of Spanish was necessary, so he added that language to his stock. Of all this language work he retains practically nothing, for he has had no occasion in late years to make use of it. He can read articles in any three of the modern languages mentioned, by keeping a dictionary close to his elbow, and he does a little reading in this way occasionally. Of conversation he is wholly incapable, except that when going home in the street cars he occasionally enjoys family gossip retailed by Germans who imagine no one in their vicinity understands the tongue. Even his meager knowledge of modern foreign languages is superior to that of 90% of the engineers with whom he comes in contact, hence his criticism of the manner of teaching languages in engineering schools, and his suggestion that this study be elective. The engineers who will really profit by it will take up this work; the "ninety and nine" who go at engineering as a vocation, and not with any idea of the study of engineering as a cultural matter, nor with the idea of being teachers, nor with any idea of doing research work, will not study foreign languages at school from choice, unless the credits gained thereby are more easily obtained than by any other method. The writer's criticism did not extend solely to the waste of time in attempting to get a conversational knowledge of a foreign tongue, but to the very poor way in which, as a rule, the study of foreign languages is taught in the majority of schools to first- and second-year students, who are obliged to take the work. It is really a device for piling up credits.

To a certain extent, the writer agrees with Mr. Boucher on the subject of the 6-year course in engineering schools. He stated in his paper a belief that engineering schools of the future in the United States will probably call for a minimum of 6 years' work. The reason for this belief is that there is a widespread demand on the part of teachers that this be accomplished. The tendency in this direction is so strong that no power on earth can prevent it from being tried. Much of the elementary work now being performed in technical schools of college grade will be attended to in technical high schools, so that in the future we shall have the Trade, the Vocation, the Business, and the Profession of Engineering, all recognized and taken care of in schools ranging from trade and high schools to the largest universities. The greater number of teachers will come from schools where the professional ideal is held, that is, these higher schools will train teachers, many of whom it is to be hoped will have considerable active practice in earning a living as vocational men before taking up teaching. The writer, in his paper, took the vocational school, corresponding to the present technical schools, as the one in which engineers should be most interested. The present 5- and 6-year courses, however, give very little, if any more, than the 4-year course in some schools, for the latter require from the students more hours per week than schools with the longer courses.

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Mr. Boucher also referred to the writer's neglect to include arithmetic as an entrance subject. The writer has taught much in evening schools, and, as a result of his experience, can say that arithmetic is taught so badly in the ordinary American school that it will be better to omit it as an entrance subject, assuming that it was completed before the student entered the high school. His experience as an instructor in evening schools, and also as an employer of office assistants and draftsmen, compels him to say that the schools of America have much to learn from the schools of Europe in teaching arithmetic. It is stated in the paper that in the first year students should devote one hour each day to going through the examples in Sanborn's "Mechanics Problems." This will give them drill in arithmetic. He mentioned also that the second-year students should be drilled on problems apt to arise every day in actual work, these problems all being arithmetical rather than algebraic.

In reply to Mr. Hunt the writer will say that it is a fact that the "tendency of the modern technical school is to become more and more narrow." This the writer wishes to counteract by his proposed arrangement of the curriculum. It will be noticed that he adheres closely to essentials throughout, merely changing the order of their introduction, with the object of broadening the minds of the men taking the work. The young man is interested in the practical rather than the ideal. He studies engineering in order that he may be enabled to earn a living. It is a mistake to cram his sciences, economics, psychology, etc., down his throat during the years when he does not and cannot appreciate them. He should be given at first the things which will make him most immediately useful to his prospective employer, to the end that the narrow-minded and undeveloped boys will be worked off by stages, leaving those whose minds develop with the school work. The humanities, therefore, come at a time when the student is maturing and the topics of the day begin to interest him. The young boy is intensely egoistic, albeit without knowing himself to be so. At about the time he reaches the age when he can vote, the problems of society begin to interest him; also, at this age, he is, as a rule, unselfish and gregarious. If he now takes up the subjects that interest men and women of standing, they will make an impression on his mind which can never be effaced, and, later, when he achieves success, he will not be considered a sort of upper-class mechanic.

Mr. Henry states that the quality of the instruction counts, rather than the instruction. It is precisely this point that the writer sought

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to bring out. He believes that the quality of instruction in the majority of engineering schools can be vastly improved. The essentials have been pretty well settled by a century of teaching. The order and the manner in which these essentials shall be imparted are now matters requiring settlement, bearing in mind that 99% of the students in engineering schools attend these schools for vocational training. When a man is drilled enough in mathematical, physical, and chemical sciences to read intelligently along the lines of his calling, he has obtained a great deal. It has been stated* that "a technical education can do nothing more beneficial for a man than to make him familiar with the best and most authoritative engineering literature." Granting that technical education gives him this much, let us add certain other broadening studies of a general nature, so that the graduate of the engineering school will be a good assistant, a well-read man, a good citizen. Those who leave before graduation will be good minor assistants, whose further development will depend on their inheritance of mentality and family environment.

Mr. Allen will find on investigation that a comparison between engineering, law, and medical schools is not at all unfavorable to engineering schools. He says "nowadays, the majority of law students first acquire a general college education, etc." It would be interesting to know where he obtained the data on which to base this assertion. A majority of the men admitted to practice as attorneys are not graduates of law schools, even to-day. A majority of graduates take courses, of two years in some States and three years in others, in schools run for profit, many of them being schools having evening sessions only. A very small percentage is graduated from schools requiring a college degree for entrance, there being less than half a dozen such schools in the United States, and these have small classes. Eminent lawyers are endeavoring to have entrance requirements stiffened, with a view to eliminating competition. Less than half a dozen medical schools require the completion of a college education before entrance, and perhaps a dozen call for two years of college work after high school. A few years ago there were 176 medical schools in the United States, but last year only 116 were reported, the recent campaign against medical schools run for profit having resulted in good. Medical men, however, are divided on the question of too severe entrance requirements. Eminent physicians and surgeons give long lists of names of men who were instrumental in advancing medical knowledge, and would never have entered the medical profession had they been compelled to complete a 4 years' college course before studying medicine. It has been stated also that few discoveries of importance have been made by men not pressed by poverty, for the temptations to ease are hard to resist when men have the means to gratify their inclinations to loaf.

* *Engineering News, Supplement*, November 17th, 1910, p. 37.

The argument is that only men backed by families of means can take a medical course if the entrance requirements are very severe. Mr. McCullough.

The movement to require longer preparation before studying law or medicine is inspired by the desire to cut down the number of practitioners. It is felt by some that, while this may eliminate a few good men, the resulting good to the profession in the improvement of the quality of the majority secured, will compensate for such possible loss. Opponents of the proposition point out that the loss possibly of another Jenner, or Harvey, or Lister is a large price to pay for securing an increased number of men fitted to shine socially, for the additional education required is not medical or surgical, but merely cultural, to the end that the members of the profession may make a good showing at "pink teas." Similar ideas prevail among men in the Engineering Profession. Some hope to have 6-year courses common, because, "there are too many engineers." Some wish to have two additional years for the purpose of enabling engineers to shine to better advantage socially. Some want a 4-year college course completed before beginning the study of engineering, for the same reason. At all events, it is seldom that the additional 2, 3, or 4 years are presumed to be spent on engineering subjects. It is pretty well settled that 3 or 4 years will suffice for the vocational studies connected with engineering, and the additional years are to be spent on the study of subjects of general interest. The writer proposes a re-adjustment of the curriculum, so that the general subjects may well come in the final years, the student being put at the vocational work as soon as possible.

Mr. Allen says "the schools will furnish that kind of education for which there is a strong demand from the students themselves." This is very pretty, but the truth is that few, if any, students entering engineering schools know what they need, still less what they want. Skilful advertising can make them believe they want anything the advertising department of the school presents for their attention. The students, that is, the undergraduates, should have nothing to say about what they want. Those who can afford to wait until the completion of a college course can do so, but the fact remains that whatever road they take to obtain a degree in engineering, on graduation they must "hunt a job." The training offered at an engineering school should be such that the graduates will be enabled to fit in quickly, wherever employed. It is known that graduates of engineering schools may look confidently forward to salaried employment shortly after graduation, whereas graduates of law and medical schools generally contemplate going into business for themselves. Their training is of an eminently practical nature. The law schools have moot courts and also require a certain amount of time to be spent in court, in the search

Mr. McCullough. for precedents, and the study of famous cases in libraries. The lecturers are nearly all eminent attorneys who lecture on their specialties. Such lectures are not as technical as lectures of engineers and are usually a guide to the critical study of some text. Medical students begin early on dissection of bodies, and from the first attend clinics in the college and assist in operations. The lecturers in the medical schools are also surgeons and physicians of standing, whose lectures are expository and non-technical guides to the critical study of texts. Lawyers, physicians, and surgeons, as well as ministers of the Gospel (whose divinity schools are vocational schools of an extreme type) are considered to be well educated, cultured men because they mingle daily with people who are well read and cultured, and cannot fail to obtain a certain degree of polish. They also have plenty of time to do considerable reading of a general character, and can discuss intelligently the questions of the day.

Law and medical students are not ignorant of conditions to be encountered in the practise of their respective professions. They go valiantly into the fight for existence, hoping to succeed and willing to stay as long as they have any staying powers. Engineering students as a rule are inexpressibly shocked after graduation when they come face to face with conditions of employment and compensation. They believe, on entering school, that the Profession is most remunerative. They find after graduation that steady positions are the exception, and that pay does not invariably increase with years of experience and increased ability. They cannot go into private practice until near middle age and after the acquirement of considerable general experience. The variety of work performed by engineers during 20 years is remarkable when one makes a study of the lives of engineers, as shown by the biographies printed in the *Transactions* of this Society. Their training as engineers is received after leaving school. The training in school is to enable them to acquire quickly, and with certainty, much that they might acquire in a practical way in offices, with the expenditure of considerably more time and energy. That is, school training for engineers is an efficiency proposition, to enable them early to be of service to their employers and of value to themselves, to the end that they may sooner mount the lower steps on the ladder of success and be engaged on work of high grade while still young and full of energy—not yet discouraged and weary because of the hard battle of life. If the application of their studies to the practical problems of their life work is taught them early at school many will secure positions with the start given in the first one or two years in school and not remain to be graduated, while others will certainly stay to get more at school.

The writer is not opposed to embryo engineers remaining 10 years in school if they wish, nor to engineers stringing an alphabet of honors after their names, representing degrees conferred in course. He will gladly welcome the day when the general public looks on engineers as being at least as well educated as men belonging to what have heretofore been termed "the learned professions." In fact, he is not certain that the day has not arrived, for engineering at present is popularly supposed to be most desirable as a profession and business, the average man looking on engineers as men who have pursued a hard course of study in school, practical, but scientific. The writer, however, is opposed to the idea that all engineering students must receive their education in the same way, and in the same number of years, regardless of ability, or inherited, or acquired characteristics. The true engineer is a student all his life, the technical school giving him merely a start. We cannot compare methods in schools for other vocations with methods in engineering schools, for in law, medicine, and theology, one path in each must be followed, while engineering is a profession to which many distinct trades contribute. Mr. McCullough.

Mr. Rogge well illustrates one point the writer might have brought out. The tendency among too many engineers is to magnify unduly the scientific and the clerical, or, as they term it, the technical, side of the work. Mr. Rogge saw that greater opportunities existed for him in getting into the business side of engineering, success following very quickly. He utilized his engineering education. It is more than likely that if he had spent several years more in school his sense of proportion would have been altered, and he would have stayed with the office instead of going out into the field as a business man.

The writer has a good friend, a consulting engineer of wide reputation, who is termed, by envious engineers, "a bluffer." There is not the slightest doubt that he would fail signally as an engineer, in the sense considered by the majority of the men contributing to this discussion, but, as an adviser on engineering matters, he is good. He was asked how he came to be so successful and said:

"The school I attended treated me badly in the way of an education, and I figured after a couple of years' work that I was doomed to be a failure in the designing end, so I took a job as timekeeper and gradually worked up until I got into business for myself as a contractor. When I failed, and failed so big that my case attracted the attention of newspapers, I found myself in demand as a practical man to advise on big construction matters, and now I am a consulting engineer and making more money than any man in my class."

The writer knows another man who also failed to get at school what he had hoped for, but who, by self study, has finally acquired all that other men received in technical schools. His success has not been marked, because he looked too much on the clerical end of engineering as the main thing, instead of looking on his education as being merely preparatory to his entrance on life.

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Some time ago, the writer received a visit from a friend who lately resigned a position, in which he received a good salary, in order that he might go into private practice. He now regrets the action. On being asked why he left, he replied that his employers kept crowding so much of their general work on him that he had no time to attend to his engineering duties, and had to leave them to young assistants. He was disgusted at having to take up many legal points and at having to bother with contractors and their troubles. His idea of engineering was to design structures. The tendency is marked among men who put many years in school to assume just this attitude, and the logical place for such men is the school room as teachers, after they have obtained some practical experience. The writer believes, and has many times expressed in writing his belief, that an engineering course is the modern ideal in education, as opposed to the classical course. It will hurt no one to take such a course, provided he can always understand that every man who takes it should not do so with the idea of being a professional engineer. As a preliminary training for business life, it ranks with a legal education. The writer likes Mr. Rogge's discussion.

In regard to the remarks of Mr. Higgins, the writer feels it necessary again to call attention to the fact that he merely proposed a re-arrangement of the curricula of technical schools so that boys with low ideals might sooner be fit to leave and go to work. Let each student feel each year that he is a little better prepared to earn a living, and if he stops going to school before he has done all the work required for a degree, he may be doing the best thing for himself and the best thing for the Profession. When the ups and downs of engineers are as well known to the general public as are the trials and tribulations of lawyers, medical men, and ministers, so that all young men who go to engineering schools face their future with wide-open eyes, such discussions as this will be out of date. The writer distinctly referred to the fact that his paper is intended to deal with the technical schools of the present day, not the university engineering schools of the future, when what is exceptional knowledge now will then be common knowledge.

Whatever Professor Swain writes is good to read, and the writer is flattered that he took time to discuss the paper. The writer does not by any means consider it a bad thing that men educated in technical schools often turn to other lines of work, and regrets that it was possible for any one reading his paper to get that impression. He does regret that the courses of study are arranged so that students seldom get to the practical side of their work until the last couple of years, this forcing them to stay in the Profession merely because they feel that their long training would be wasted. Parents, who pay the bills, always feel that way, so the courses of study might be arranged to give the young men practical training from the start. It has been asked what specifications an engineer might propose. They have been pretty well stated by Professor Swain: "He wants a man who is faithful, who is of good character, conscientious, who can think straight, who will not be anxious to stop work as soon as the bell rings, who will be loyal to his employer, who has 'gumption,' and who can meet emergencies." He might add that the school should also take considerable pains to make the students understand the actual conditions attached to engineering employment and the compensation therefor, the importance of living on half the pay when earning, to understand that employers have nothing against young graduates as such, but because few of them are worth their small pay for several months after leaving school, some not for a year or more. Employers also want men skilled in common arithmetical computation and with the ability to make neat drawings and do decent lettering; these, in addition to all the qualities of manhood mentioned by Professor Swain and necessary as well in other lines of business. Young men are not intrusted with important work, so their education should fit them to do well the small and comparatively unimportant things their employers put them at. A careful reader of the paper should see that the writer lays considerable stress on the studies enabling men to mix well with the world. Mr. McCullough.

A high standard for the Engineering Profession is very well, and the writer is as keen for it as any engineer, but the paper he presented was from the point of view of the more than 90% of students who take engineering courses for their purely vocational, and not for their cultural, value. These green young men and boys enter a school to study engineering with the intention of earning a living at engineering work, and do not know what it implies or what the real opportunities are. At the end of the freshman year they must select some specialty, still ignorant, for the freshman year is merely an extension of high school and there is seemingly no tie in it to the life of an engineer. A month ago a young man called on the writer for advice as to his future. He entered a State university for a college course and met a boy who persuaded him to enter the college of engineering. This was the first time he knew that engineering did not necessarily mean the running of an engine. He remarked that he could see little difference between the freshman work and the senior year in high school, and drifted along unthinkingly until spring when he was suddenly made aware of the fact that the university gave eleven distinct engineering courses, and he must make a selection of a specialty. He still knew no more about the calling of the engineer than he did on leaving high school. His parents could not help him, but his indecision was settled by a series of social events of the

Mr. McCullough. eleven engineering societies, who were engaged in "rushing" freshmen. The mining society gave what he called the "swellest" reception and entertainment and had the best floats in the annual college parade. The career of John Hays Hammond was at that time attracting considerable newspaper and magazine attention, so the boy entered the mining school. This may sound far fetched, but he states as a fact that he took the sophomore and junior work in the mining department without seeing a mine. A requirement of the school is that students must spend not less than 3 months in some vacation in actual mining work, in order to be eligible to enter the senior class and obtain the degree in mining. In his first vacation he helped the county surveyor near home. In the second vacation he was a draftsman in the office of a structural engineer. This past summer he had to do mining work or be unable to register this fall as a senior, so he went into a mining district to seek employment. He worked for 3 months, but to the last day was unable to rid himself of a disagreeable feeling in the pit of his stomach when going down a shaft. He was always impressed with a feeling of insecurity when in the workings, and the number of accidents he witnessed were not reassuring. On top of the ground he is all right, but he hates to think of spending his life in mines. He was advised to complete his course of study and get rid of the feeling that since he studied mining engineering he must of necessity follow that as a profession. His training in surveying, drafting, mathematics, physics, and chemistry, will enable him to be a good assistant in the office of an engineer or manufacturer, which, after all, is the most that a technical school should expect to give, the technical school, it must be remembered, being something different from a high-grade engineering school attached to a university and headed by men like Professor Swain.

Professor Swain says: "It is impossible for a man who has not tried to teach to draw up a curriculum which will work well; he almost always forgets that the problem of engineering education, or of education in general, is not an engineering problem, but a human problem." The writer begs to state that he has not only tried to teach, but is rated as a successful teacher. He has taken classes abandoned by professional teachers, and greatly increased them in number because he understood the men with whom he was dealing, their problems encountered in trying to earn a living, their object in studying at night after working all day, the best methods of handling them so as to inspire interest in the subject and hold it to the completion of the work. He has also been successful in coaching young men unable to follow intelligently their paid teachers in college and technical schools, boys who would otherwise have been "flunkers." In his paper he endeavored to deal with the problem of engineering education as a human problem, the subject being discussed as a vocational, and not as a purely educational proposition. He has tried to suggest that the instruction be imparted somewhat more practically in the first two years, in a human and humane manner. The writer must remind Professor Swain, as he has other men who have presented discussions, that he merely proposed a change in the order of studies and did not propose a brand new curriculum. Mr. McCullough.

In reply to Messrs. Fuller and Buerger, the writer must call their attention—as he has called the attention of others preceding them in the discussion—to the paper. He fails to find anything in it to lead any one to believe that he advocates a narrow training or that he decries education of the proper kind. He simply attempts to rearrange the curriculum, omitting nothing of value, adding much of value, and postponing to the reasoning years subjects deemed "cultural"; leaving the study of economics, history, literature, sociology, etc., to minds trained not to memorize, but to think.

Teachers uniformly resent suggestions from practising engineers and from employers of engineering graduates, claiming that such suggestions have a narrowing tendency, and that men not teachers do not put the proper "cultural" value of education to the front. This is not borne out by the facts. A study of discussions on engineering education, from the time such discussions commenced, will show that the practising engineer has been more instrumental than the teaching engineer in having more attention paid to general subjects. The practising engineer laughs at the long array of specialties listed in catalogues of engineering schools, and knows, as the result of actual experience in winning a living, that a few fundamental things well taught are sufficient; but they must be well taught. The teachers, each one anxious to magnify his importance in the faculty and gain glory and higher pay, are the men responsible for the narrowing of the curriculum. Teachers, by pushing special courses, which the bewildered freshman must consider, stultify their remarks about general education and the cultural value of education. Professor Fuller says:

"In talking with some of his own students the speaker has noticed a greater inclination to take general work in the latter part of the curriculum than in the first. If given in the first part, it is thrust upon them; if available later, many will take it willingly. The speaker has heard practicing engineers suggest such an arrangement."

If this be so, then why not try it?

Without wishing to appear to be a critic of teachers, for he also teaches, because he likes it and teaches a class of men who come voluntarily to get the work, the writer must say that no class of men is less tolerant of suggestion and apparent criticism, than teachers, beginning with the kindergarten grade. This is for the reason that teaching is a vast organized profession, fettered with precedent and hampered by tradition. These remarks must be softened by the state-

Mr. McCullough. ment, that with all the criticism of the teaching class indulged in by people who must employ the product turned out of institutions of learning, the greatest changes and improvements in teaching methods have come from the ranks of the teachers. However, there is a deadening influence at work tending to weaken those who teach continuously many years. For this reason, the writer is greatly in favor of teachers in technical schools being employed on practical work, and thinks there should be a greater amount of practical work demanded of them. Good teachers should be given leave of absence at stated times, under full pay, so that they may go into the ranks of engineers, to the end that the deadly monotony, inherent in all large organizations and classes, shall not stunt their minds.

Professor Fuller asks, with others, for a specification for the preparation of engineers' assistants. It has been given already in this closure, as well as in the paper. The writer nowhere stated, nor did he imply, that the product of the "engineer factories" should be guaranteed, as some of the gentlemen who have discussed the paper facetiously remarked. A reference again to the paper is suggested. The reason for asking that the wishes of the employer be more carefully considered has been sufficiently dealt with in the paper and in this closure.

The writer agrees with Mr. Buerger that the best training is the most broad, and that a division into specialties is to be deplored, as far as undergraduates are concerned. Employers, however, are not willing to give all the practical training so essential. There are too many thousands of graduates turned out annually from technical schools to compel the employer to waste much time with the unfit and incompletely trained. A three-line advertisement in the Sunday edition of any good daily paper will suffice to fill the mail box to overflowing with applications for work. Short shrift is given those who do not take hold quickly. Many who might otherwise have been successful are doomed to wander for many years from job to job, because of the false view of life obtained in the institutions supposed to be created for the purpose of supplying the demand of the industrial world for trained workers. The technical school is assumed to exist for a particular purpose, and it does not fulfill its mission if the majority of graduates fail to meet with as much success as the average man.

The writer endorses most heartily all that Mr. Buerger says, beginning with the words, "The ordinary school lecture is an abomination," and continuing to the end of his discussion, which should be taken to heart by every teacher, every practising engineer, and every employer of the product of engineering schools. Make the boys work hard from the start. Teach a smaller number of subjects at one time if necessary, to carry out the ideas expressed in his two sentences relating to methods of teaching. The employment of older students to assist the teacher is excellent, as the writer has found in his own teaching experience, for it helps every one. A man learns best when he has to teach, and the student is inspired when he works with his teacher, instead of trying to do what he is told to do, with occasional guidance from one who assumes a superior attitude. Mr. McCullough.

A fitting end to this discussion is the following:*

"*Educating the Educators.*—The University of Cincinnati was one of the first in this country to apply continuation school methods—giving a pupil shop practice under actual commercial conditions, along with textual instruction. Dean Schneider, of the engineering college, has made some interesting confessions of the reflex action upon the university faculty of this practical shop training. He says:

" 'We learned the first year, and have had it verified each year since, that the shop will spot a yellow streak in a man before the university even suspects it. An attempt to sneak through spoiled work is never a great success there. We, at the college end, soon found our work under scrutiny and criticism from a source that does not hesitate to scrutinize and criticise. We are brought face to face with the failure of a university department as we never are in our four-year courses. A student, let us say, has finished successfully his work in physics. Some day he does a fool thing in the shop which indicates that he knows very little about the subject. When you confront him with the fool thing, and with the fact that he should have known better because he had been taught the theory governing it, you find his grasp upon the theory to be very feeble.'

"Practical education will teach the teachers. We imagine it would not be a bad thing in every university if pupils and instructors, pleasantly loafing through their four-year literary courses, were periodically checked up by some hard-and-fast test drawn from actual life outside the campus, whereby they could discover exactly how efficient their processes were."

* Editorial from *The Saturday Evening Post*, October 5th, 1912.

MEMOIRS OF DECEASED MEMBERS

WILLIAM BILLINGS CLAPP, M. Am. Soc. C. E. *

Died December 27th, 1911.

William Billings Clapp was born in Conway, Mass., on April 11th, 1861. In his early boyhood his parents migrated to Southern California and were among the first group of families in the "Indiana Colony" that settled on Pasadena lands. The story of this colony forms in large measure the history of modern Pasadena.

Mr. Clapp's engineering education was largely acquired in the field, and the field of California was at that time, as it is to-day, unusually rich in opportunities for practical engineering instruction. In 1882 he was Assistant in public land subdivision surveys in Northern California. From 1883 to 1886 he was Assistant Engineer in charge of the construction of the Los Angeles and San Gabriel Valley Railroad. Subsequently he was Assistant Engineer for the Los Angeles and Salt Lake Railway Company, and in 1889 engaged in private engineering work at Seattle, Wash.

Returning to California in 1891, he became Engineer and Roadmaster for the San Gabriel Valley Rapid Transit Company. In 1893 he engaged in general practise in Pasadena, Cal., and in 1894, and for six years subsequent thereto, he was City Engineer in Pasadena, during which period that municipality began its transition from a rural settlement to the modern city of to-day.

In the course of his service, Mr. Clapp was identified with the radical improvements and changes that necessarily precede and are fundamental to modern city development. From 1901 to 1903 he again engaged in private practise, giving a part of his time to the water resources investigations of the Geological Survey. On May 5th, 1903, he was appointed an Assistant Engineer in the Geological Survey, and from that date until his final illness he was identified with the hydraulic investigations of that bureau in California. During the last seven years of the period he was its District Engineer for that State.

It appears that, in the hydraulic branch of the Profession, Mr. Clapp found the work for which he was best fitted by inclination and experience. He had grown up in a region in which the agricultural possibilities are limited only by the available water supply and the skill with which it is conserved and applied. He had encountered, during his entire professional life, all the difficult engineering problems that beset an arid country. Even while engaged in other

* Memoir prepared by M. O. Leighton, M. Am. Soc. C. E.

lines of engineering work, he had been constantly in touch with water-supply developments and the problems in relation thereto. Therefore, when he entered the Federal service he was mature in hydraulic matters, and his work was productive from the very beginning. Within a short time his reports and opinions were regarded as authoritative, and were accepted as *prima facie* evidence in the Courts of California.

Although, by reason of the distance from his headquarters to New York City, Mr. Clapp rarely found an opportunity to take part in any of the meetings of the Society, his interest in it was profound, and he took singular pride in his membership. His principal contribution to the publications of the Society* was a study of the Sacramento River flood of March, 1907, written in collaboration with E. C. Murphy, M. Am. Soc. C. E., and W. F. Martin, Assoc. M. Am. Soc. C. E.

The study and analysis of this disastrous flood was attended by unusual complications. In few, if any, recent inundations of this kind have consequences of greater magnitude been dependent on the correct interpretation of obscure evidence. Sacramento floods have been a perplexing problem throughout California history, and have been investigated repeatedly by able engineers in search of suitable measures of relief. A short time before the flood of 1907 extensive plans had been proposed by a competent Board of Engineers for reclaiming the enormous agricultural values of the Sacramento Basins and preventing further flood destruction. The study by Mr. Clapp and his associates demonstrated that those plans provided for less than half the capacity necessary to achieve the desired purpose. Substantial evidence of the accuracy of the work is contained in a recent report of a Board of Army Engineers, designated, under the authority of an Act of Congress, to make a complete investigation and submit a report on a plan for the control of floods in the Sacramento and San Joaquin Valleys. This report contains the following statement:

"It is thought that the estimates of Messrs. Clapp, Murphy, and Martin should be followed very closely in determining the necessary channel widths, and their maximum is assumed with certain allowances for flattening of the flood wave in passing down the improved channels."

In the progressive development of the State of California, so dependent on the wise utilization of its water supplies, it is evident that, as valuable as Mr. Clapp's work now appears, it has not begun to accomplish that which it finally will. In the future, more than in the present or past, the great hydraulic developments of that State will be guided in large measure by the observations and fundamental records that this quiet but forceful man has given to the people.

* *Transactions*, Am. Soc. C. E., Vol. LXI, pp. 281-330.

Mr. Clapp died at his home in Pasadena, Cal., on December 27th, 1911. During the last half year of his life he was in the grasp of a hopeless malady, throughout the course of which he amply demonstrated the steadfast characteristics that marked the whole of his useful career. He was one of those men whose best qualities are exposed only to those who achieve their entire confidence. Without pretense or display, he was content to allow his acquaintances to base their personal appraisal on those of his qualities that could withstand the scrutiny of close association. That he had, under such a social and professional creed, gathered to himself a host of genuine friends is the most potent evidence that can be cited concerning his personal merits. Because he was a real out-of-door man, the majority of his friends were those whom he had encountered somewhere on the trail, where men quickly find their level, and friendship is a natural consequence among those who make good.

Mr. Clapp was elected a Member of the American Society of Civil Engineers on December 6th, 1905.

BENJAMIN DOUGLAS, M. Am. Soc. C. E.*

Died November 12th, 1911.

Benjamin Douglas, son of Judge Samuel Townsend Douglas and Elizabeth (Campbell) Douglas, was born at Detroit, Mich., on December 10th, 1859. Most of his life was spent on Grosse Isle, a large island in the Detroit River, where his father had built a home in 1859.

His early education was given to him by his mother, who also prepared him for entrance into the University of Michigan. He was one of a small class in Civil Engineering in that University, and was graduated in 1882.

On leaving college Mr. Douglas became Assistant Engineer in the Detroit Bridge and Iron Works, where he remained until February, 1885. He then became Engineer of Bridges on the Michigan Central Railway, which position he retained for twenty years. Mr. Mock, of that Railway Company, says:

"The notably successful way in which he met the problems incident to the rebuilding of bridges with the least interruption to traffic, and the entire reliability of his work, gave him a high standing among the Bridge Engineers of the United States and Canada.

"He replaced the bridge over the Grand River, in 1891, by moving out the old bridge, one span at a time, and moving in the new span from its temporary support alongside, and, on the last two sections of 154 ft. each, accomplished the substitution, with the bridge ready for use, inside of 26 minutes. This was one of the first bridges, if not the first, to be so handled, and I think it is a record unbeaten to this day."

Mr. Douglas also had charge of the rebuilding and strengthening of the Cantilever Bridge at Niagara. In 1894 he designed the first bridge with a solid I-beam ballast floor, a construction which is far superior to any except concrete.

"To him is due, in a very large measure, the success attending the building of the Detroit River Tunnel, for which he represented the Michigan Central Railway. This tunnel is a new departure in subaqueous tunnel building, and its problems called forth a large number of designs—one such was made and patented by Mr. Douglas."

On the completion of the tunnel Mr. Douglas opened an office as Consulting Engineer, and, in that capacity, went to Southern Brazil, where he was engaged on the Soro Cabaña Railroad. This road was being reconstructed and extended, and presented many bridge and viaduct problems. His work there was almost completed, when, by a misstep on the temporary flooring of a high viaduct, he was precipitated to the ground below and almost instantly killed. Through the kind

*Memoir prepared by the Secretary from papers on file at the House of the Society and from information furnished by his family.

offices of his friends in Brazil, and of the officials of the Soro Cabaña Railroad, after a service by Bishop Kinsolving in the little church of Santa Maria, his remains were sent to New York, whence they were conveyed to his home to be interred beside his parents and grandparents in the old cemetery in Detroit.

Mr. Douglas' life was a peculiarly harmonious one. He found in his beautiful home and farm congenial occupation for all the time not given to his profession. His devotion to his parents was repeated in the affection of his own family life. A man of sterling integrity, his modesty and reserve concealed to some extent his abilities in his profession and the fine qualities which he possessed, but these were gradually understood and appreciated by those who came in contact with him or his work. He chose that which was best in life, and worked steadily toward it.

He was a Past-President of the Detroit Engineering Society; a member of the Michigan Engineering Society (and in November, 1911, was appointed on the Legislative Committee of that Society); a Member of the American Railway Engineering and Maintenance of Way Association, and a member of Tau Beta Pi, the honor society of graduate engineers in the University of Michigan.

Mr. Douglas was elected a Junior of the American Society of Civil Engineers on June 1st, 1887, and a Member on January 2d, 1890.

JULIEN ASTIN HALL, M. Am. Soc. C. E.*

Died January 12th, 1912.

Julien Astin Hall was born in Richmond, Va., on January 29th, 1858. He began his career in March, 1880, as Chainman and Rodman on the preliminary and location surveys for the Danville and New River Railroad (now the Danville and Western Railroad), in Virginia, and the Dan Valley and Yadkin River Railroad, in Virginia and North Carolina. In January, 1881, he was made Engineer in general charge of construction and revision of location of the former road, remaining in that position until July, 1882. He was also employed, at the same time, as Resident Engineer of the Danville, Mocksville and Southwestern Railroad, in Virginia and North Carolina.

In July, 1882, Mr. Hall was appointed Assistant Engineer for the Richmond and Danville Railroad, in charge of construction and general surveys. In December, 1882, he re-entered the service of the Danville, Mocksville and Southwestern Railroad, as Engineer in charge of re-location and general surveys. He retained this position until January, 1884, when he was engaged as Assistant on change of location of the main line of the Richmond and Danville Railroad, at State Line, Virginia and North Carolina.

From June, 1884, to September, 1886, Mr. Hall was employed as Engineer in general charge of maintenance of roadway, bridges, and buildings, for the Danville and New River Railroad. He was then employed as Assistant Engineer on the Richmond and Danville Railroad, with headquarters at the Chief Engineer's office at Washington, D. C. In October, 1886, he received the appointment of First Assistant Engineer with the same road, later being made First Assistant to the Chief Engineer.

Mr. Hall left the Richmond and Danville Railroad Company to enter the service of the Erie Railroad, with headquarters in New York City. After leaving the Erie Railroad, he was engaged with Rinehart and Dennis, Contractors, on the "Big 4," at Danville, Ill., and as Engineer for Grant Wilkins, Contractor, on the Tennessee River Bridge, at Knoxville, Tenn., for the Marietta and North Georgia Railroad. He also made surveys for a number of water-power projects in the South.

In 1905, Mr. Hall, who was then with the Southern Railroad, with headquarters at Washington, D. C., was compelled to give up active work on account of failing health. He moved to Wenonda, Va., where he was engaged in consulting practice until his death on January 12th, 1912.

Mr. Hall was elected a Member of the American Society of Civil Engineers on June 5th, 1889.

* Memoir prepared by the Secretary from information on file at the Society House.

ARTHUR POWIS HERBERT, M. Am. Soc. C. E.*

Died June 16th, 1912.

Arthur Powis Herbert was born in Philadelphia, Pa., on August 30th, 1855. He began work as a Rodman on the Colorado Central Railroad in 1872, and was also employed on the Union Pacific and Utah and Northern Railroads. In 1876-77 he was with the American Dredging Company, of Philadelphia, and in December of the latter year went to Brazil with the ill-fated Collins Expedition on the construction of the Madeira and Mamoré Railroad. On his return, in 1879, he was employed on the engineering force of the Pennsylvania Railroad as Assistant Engineer on the Valley Creek Improvement, and in charge of the Coatesville and Pomeroy Improvement.

He was one of the railroad pioneers of Mexico, going there in November, 1880, under the late W. C. Wetherill, M. Am. Soc. C. E., Chief Engineer, as one of the engineers of the Mexican National Construction Company (Palmer-Sullivan Concession) on location and construction across the mountains from Mexico City to Toluca. In 1886, he was for about a year in charge of the extension of the Oroya Railroad at Cerro de Pasco, Peru. Returning to Mexico, he had charge of the location of the Guadalajara Branch of the Mexican Central Railroad, under the late John E. Earley, M. Am. Soc. C. E., and also of parts of the San Luis Potosi and Tampico Divisions.

Later, he again entered the employ of the Mexican National Construction Company, for which he made various surveys and locations and finally, under the late H. H. Filley, M. Am. Soc. C. E., Chief Engineer, located and built the portion of the Colima Division of the Mexican National Company's line between Armeria and Colima. Subsequently, he was made Superintendent of this branch, holding the position for about twenty years or until it was taken over by the Mexican Central Railroad Company and merged into one of the other divisions of that Company.

Mr. Herbert was well known in Mexico and South America. He was a careful and successful locating engineer, showing such rare judgment in his choice of lines that his final location seldom varied more than a few meters from the preliminary. His sterling worth was universally appreciated by his superiors, by whom he was trusted implicitly; his genial disposition and his strict and impartial justice won for him the friendship, admiration, and high esteem of associates and subordinates.

He died suddenly, from apoplexy, on June 16th, 1912, at his home in Colima, Mexico, leaving a widow and one daughter.

Arthur Powis Herbert was elected a Member of the American Society of Civil Engineers on September 5th, 1888.

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* Memoir prepared by Caspar Wistar Haines, M. Am. Soc. C. E.

JAMES BREADING HOGG, M. Am. Soc. C. E.*

Died June 4th, 1912.

James Breading Hogg, first son of John T. and Caroline (Austin) Hogg, was born on December 15th, 1857, near Prittstown, Bullskin Township, Fayette County, Pa. Soon after his birth his family moved to what was then New Haven, on the west bank of the Youghiogheny River, now known as the West Side, Connellsville, Pa., where he spent the first twenty-seven years of his life.

He was graduated from Lafayette College, Easton, Pa., with the degree of Civil Engineer in 1881. He was a member of the Lafayette College baseball team, and also played on one of the early baseball teams of his home section.

After graduation he commenced his engineering career, under the late Joseph H. Paddock, M. Am. Soc. C. E., as an Instrumentman on the construction of the Pittsburgh, McKeesport and Youghiogheny Railroad, between Layton and New Haven, Pa., and the Dickerson Run Branch of the same railroad. He was also on the construction of a highway bridge across the Youghiogheny River at Dawson and a railroad bridge over the same river at Broad Ford. At that time the writer was also a member of the Engineering Department of that railroad, and it was then that his acquaintance with Mr. Hogg began.

In 1886 Mr. Hogg went to Puget Sound, where he served as Assistant Engineer on the Cascade Division of the Northern Pacific Railroad, having charge of 10 miles of heavy construction work. On its completion he was associated with the City Engineer of Seattle, and for two years had charge of that office. He was Assistant City Engineer of Seattle at a time when the city was being laid out along progressive lines which necessitated extensive changes in grades and street lines. The beauty of Seattle's streets to-day is due in a large measure to the intelligent and able engineering work of Mr. Hogg.

Later, he was Chief Engineer of the Port Townsend Southern Railway, and for two years directed the exploration, location, and construction of its line. Then he became Assistant Engineer for the Oregon Railroad and Navigation Company, and constructed 15 miles of its track. He was also employed in the Washington State Engineering Department at various times.

During a period of industrial depression, when railroad construction was suspended, Mr. Hogg was for two years County Treasurer of Jefferson County, Washington. He returned to the East in 1900, and entered the service of the H. C. Frick Coke Company, at the time of the spectacular entry of Andrew Carnegie, the ironmaster, into the railroad world. Here he planned a railroad through the Connellsville coke region which connected all the plants of the H. C. Frick Coke

* Memoir prepared by Emile Low, M. Am. Soc. C. E.

Company. He also made explorations across the Allegheny Mountains for Carnegie's proposed railroad to the Atlantic Coast. Later, he was Engineer in Charge of the surveys for the same company and became Division Engineer of the Northern coke field.

In 1902 Mr. Hogg was transferred to the Frick interests in West Virginia, and made topographical surveys in the Tug River District of McDowell County in that State. Completing that work, he returned to Fayette County and opened an office in Connellsville, Pa., for general consulting practice. He also maintained offices in Uniontown, Brownsville, Scottdale, and Pittsburgh, all in Pennsylvania. When death halted his work he was serving his fourth term as County Engineer of Fayette County. He was also Borough Engineer of Connellsville, Scottdale, and Everson, and previously had been Borough Engineer of Coraopolis, Bentleyville, and West View. In addition, he made comprehensive sewer plans for Connellsville, Uniontown, Scottdale, Brownsville, West Brownsville, and South Brownsville.

Mr. Hogg was one of the best known civil engineers in Western Pennsylvania, and ranked high in his chosen profession. He lived an active life, gained a wide experience in all departments of engineering, and built up probably the most extensive engineering business in that section of Pennsylvania. Realizing that municipal sanitation was to become an important factor in that State he specialized diligently in that line of work. Although stricken in the prime of manhood, he had already attained a degree of prominence as a sanitary engineer that caused his counsel to be sought by many municipalities in Western Pennsylvania.

His first work was the preparation of a comprehensive sewer plan for Connellsville, and the map which he completed was one of the best ever submitted to the engineers of the Pennsylvania State Board of Health. He was frequently called to Harrisburg to consult with the State sanitary engineers, and his judgment was highly respected.

Mr. Hogg had great faith in the future of Connellsville. He believed, not only that the town would grow, and grow rapidly, but was firm in his conviction that eventually there would be a civic awakening resulting in Connellsville being not only a prosperous city, but a more desirable place in which to live. He believed in the "city beautiful" movement, not in the sense that has made such a sentiment a reproach, but that a serious, concerted effort should be made to beautify the streets, the houses, and surroundings. He believed that Connellsville was neglecting a wonderful opportunity in failing to capitalize its river, to beautify the river front, and, if possible, dam the stream and create of it something useful to the community. He could not understand why such a manifest advantage was neglected, or why more interest was not exhibited in the movement to utilize the river (Youghiogheny), which he, with a few others, fully realized.

Mr. Hogg did not hesitate to back his faith with works. The beautiful East Park Addition to Connellsville will ever be a monument to his belief in Connellsville and his efforts to make it more attractive. For years "Hogg's pasture" had furnished grass for the cows of the community. Its hills rose beyond a steep ravine which seemed a perpetual obstacle to development. In 1904 Mr. Hogg determined to lay out a new addition. It was not primarily a land selling scheme. Mr. Hogg had other plans, and, in co-operation with the John T. Hogg estate, proceeded to make an investment that seemed out of proportion to the possible returns within a generation. An immense steel bridge was constructed from Baldwin Avenue across the hollow through which Connell Run winds its way to the river. Beyond that point Mr. Hogg set his engineers at work. A boulevard was laid out through the Hogg estate as far as the Reidmore Road. Once the lines were laid, the road was built and was one of the most attractive driveways in that section.

At the time the East Park Addition was conceived, Mr. Hogg's home was in Uniontown, although his engineering offices were in Connellsville. He immediately awarded a contract for the erection of a fine dwelling in the new addition. For a year or more it loomed alone in the heart of the barren pasture. Then came other residences, and before long East Park Addition had "arrived."

The lots were sold under certain building restrictions which encouraged the erection of attractive homes. Where six years ago there was but one home, and that occupied by Mr. Hogg himself, for he moved there soon after its completion, there is to-day quite a colony of attractive residences, and others are contemplated. With characteristic public spirit Mr. Hogg dedicated the boulevard to the public. This was named Will's Road, in memory of his brother, the late William A. Hogg.

Mr. Hogg was elected a Member of the American Society of Civil Engineers on October 3d, 1906. He was also a Member of the Engineers' Society of Western Pennsylvania, the Engineers' Society of Pennsylvania, and the American Mining Congress of Denver. Aside from his business, Mr. Hogg had time to give to other matters. He was a member of the Chamber of Commerce of Connellsville, and one of its Directors. He was also a member of the Uniontown Country Club and the Laurel Club. In politics he was a staunch Republican, but in no way a politician. In religion he was an Episcopalian.

On December 22d, 1903, Mr. Hogg was married to Miss May Reid, daughter of the late William T. Reid. He is survived by his widow and also his mother and four sisters.

Mr. Hogg died on June 4th, 1912, and was buried in Allegheny Cemetery, Pittsburgh, Pa., where his father is laid at rest.

The following resolutions, passed by the Connellsville Chamber of Commerce, show the respect in which he was held:

"James Breading Hogg, member of the Chamber of Commerce of Connellsville, and one of its most valued, capable, and useful directors, was possessed of those sterling qualities which differentiate men of worth and achievement from those of sham and pretense; of those enviable characteristics which are the outstanding traits of men of mark and usefulness in a community's life; of that temper of mind and fulness of heart which drew others to him by the ties of friendship, respect, and esteem, and of that tenderness and gentleness which enshrine his memory in the most affectionate remembrance of those who loved and were loved by him.

"He attained conspicuous rank in his chosen profession because of thorough preparation, close application, fidelity to the interests he served, and the high ideals by which he was constantly stimulated and impelled.

"As one of Connellsville's First Citizens, the disinterested, zealous, public-spirited services he rendered in the development and advancement of the material interests, and the enhancement of civic virtue, of the whole community, entitle him to enduring honor and a people's gratitude.

"Proving his faith in the future growth and greatness of his native town, by his own works and efforts to arouse a civic awakening that would be active, progressive, and permanent, he set an example worthy of emulation by every citizen who should seek, as unselfishly as did he, to make Connellsville a better home for the present and all succeeding generations.

"As an earnest and sincere, though by no means adequate, measure of the Chamber's appreciation of the life services, character, and attainments; as a meed of praise of the deeds he wrought, and in highest respect, admiration, and regard for our lamented associate and coworker, these sentiments are recorded, by order of the Chamber, this sixth day of June, A. D., 1912."

EDWARD HENRY KEATING, M. Am. Soc. C. E.*

Died June 17th, 1912.

Edward Henry Keating was born at Halifax, Nova Scotia, on August 7th, 1844. He was the fourth son of William Henry Keating, Barrister, who, for many years, was Deputy Provincial Secretary of Nova Scotia.

Mr. Keating received his education at the Free Church Academy, Halifax, and at Dalhousie College. His studies had been directed with a view to taking up Architecture as a profession, but he was soon engaged in surveys, and studied Engineering under George Whitman, Civil Engineer, Provincial Government Engineer of Nova Scotia.

For three years Mr. Keating was employed on the surveys, location, and construction of the Truro and Pictou Railway, and was also engaged as Chief Draftsman of the Windsor and Annapolis Railway. At the close of 1867, he entered the employ of the Intercolonial Railway and was Assistant Engineer on some of the heaviest construction, remaining in this position until the road was nearly completed.

In the spring of 1872, Mr. Keating was appointed Division Engineer in charge of exploration on the Canadian Pacific Railway surveys, but resigned at the end of the same year to become City Engineer and Engineer of the Water-Works of Halifax. He was also Resident Chief Engineer of the Halifax Graving Dock, in its time the largest graving dock on the American Continent. He remained in Halifax until the end of 1890. During his residence there, he also designed water-works for Truro, Windsor, and Dartmouth, Nova Scotia, and for Moncton, New Brunswick.

For two years, 1891 and 1892, Mr. Keating was City Engineer of Duluth, Minn., where he designed extensive improvements for the water-works. He was then offered and accepted the office of City Engineer and Engineer of the Water-Works of Toronto, Ont., and soon reported on a comprehensive plan for the improvement of its water-works. This plan was considered too expensive at the time, but was afterward carried out. He did important improvement work in Toronto Harbor, and also in Keating Channel which was named after him. In 1898, Mr. Keating resigned as City Engineer of Toronto to become General Manager of the Toronto Street Railway, which position he held for six years.

In 1903, he was appointed Chairman of the Royal Commission to report on the construction of a graving dock for the City of Montreal. He also acted, at various times, as Expert Advisor on the water-works and sewerage systems of Ottawa, Hamilton, Victoria, B. C., etc.

* Memoir prepared by W. H. Breithaupt, M. Am. Soc. C. E.

In 1904, Mr. Keating went to Mexico where, for the greater part of two years, he was Engineer and Manager of the construction of the street railway and power and lighting plant of the Monterey Railway, Light and Power Company, and of the Monterey water-works and sewerage systems. Of these properties he remained Advisory Engineer for some years after his return to Toronto.

For the last six years of his life, Mr. Keating was active as a Consulting Engineer in general practice, being engaged for a greater part of his time as Arbitrator in valuation proceedings, for which his sound judgment and absolute sense of justice admirably fitted him.

Mr. Keating was elected a Member of the Institution of Civil Engineers in 1878, and had been a Member of the Council since 1911. He was President of the Canadian Society of Civil Engineers in 1901, having long been a Member and Member of the Council of that Society. He was also a Fellow of the Imperial Institute, a Member of the Engineers Club of Toronto, of which he was one of the organizers, and of the Toronto and York Clubs.

In February, 1907, he was appointed a member of the Board of Examiners for Professional Degrees in Engineering, by the University of Toronto, and was Chairman of this Board until early in 1912, when, on account of ill health, he was compelled to resign as Chairman, though he continued as a member until his death on June 17th, 1912.

Mr. Keating was elected a Member of the American Society of Civil Engineers on June 7th, 1882.

LEWIS KINGMAN, M. Am. Soc. C. E.*

Died January 23d, 1912.

Lewis Kingman was born on February 26th, 1845, at North Bridgewater, Plymouth County, Mass., near the landing of the Pilgrim Fathers. He was the first of six children born to Isaac Kingman and Sibil Ames. He died in the City of Mexico, of pneumonia, on January 23d, 1912.

His early youth was spent at home. His education was in the common schools, supplemented by three and one-half years in Hunt's Academy, a local institution of merit, in which he finished his work in the winter of 1861.

In September, 1862, he entered on a three-year course in Civil Engineering with Shedd and Edson, of Boston, Mass. For this instruction he was to pay $100 per year. During his course with Shedd and Edson, which was reduced to one and one-half years by mutual consent, he lived at home, twenty miles from Boston *via* the Old Colony Railroad, and on this road he made daily round trips on student's tickets purchased at the rate of $52 per year.

During the course under Shedd and Edson he had a varied and valuable experience in municipal and corporate engineering.

For about four years, 1864-1868, Mr. Kingman was engaged in the Pennsylvania oil fields, with a brief period of about two months in engineering work in New York City. He was in the Pennsylvania oil fields when, in June, 1866, oil sold for $7 per barrel, and at 90 cents per barrel less than one year later.

In 1868 he went to St. Louis—attracted by railway building then in progress—and on July 13th engaged with the Eastern Division of the Atlantic and Pacific Railroad. He was with the Eastern and Western Divisions of that enterprise, in various capacities on construction and surveys, until October, 1871, when his party was disbanded at Las Vegas, N. Mex.

It is interesting here to note that, during this period, Mr. Kingman and the late James Dun, M. Am. Soc. C. E., had adjoining residencies on the construction of the Atlantic and Pacific Eastern Division, and that, later, Mr. Dun was Chief Engineer of the Eastern Division, known as the St. Louis and San Francisco Railroad, with headquarters at St. Louis, Mo., and Mr. Kingman was Chief Engineer of the Western Division, known as the Atlantic and Pacific Railway, with headquarters at Albuquerque, N. Mex.

From the latter part of 1871 to the middle of 1877 Mr. Kingman was engaged on a survey from Kit Carson, Colo., to Cimarron, N. Mex., in the interest of the Maxwell Land Grant Company, owner of a large

* Memoir prepared by A. A. Robinson and C. A. Morse, Members, Am. Soc. C. E.

Spanish land grant located in Colorado and New Mexico, and on Government land surveys, and other engineering work and commercial business.

In June, 1877, Mr. Kingman began work on the Atchison, Topeka and Santa Fé Railway under A. A. Robinson, M. Am. Soc. C. E., Chief Engineer of that Railway, and was employed in various localities, on mountain surveys and construction, in Colorado, New Mexico, and Arizona, until July, 1880, at which time he was on the Atlantic and Pacific Railway (an auxiliary to the Santa Fé) where he remained until April, 1883; he was appointed Chief Engineer of this Company on January 1st, 1882, succeeding the late H. R. Holbrook, M. Am. Soc. C. E.

Mr. Kingman then (April, 1883) accepted the position of Chief Engineer of the Northern Division of the Mexican Central Railway, under Mr. Thomas Nickerson, President, and served in that capacity until June 1st, 1884, during which time he constructed 469 miles of the Mexican Central Railway.

In July, 1884, Mr. Kingman returned to the service of the Atchison, Topeka and Santa Fé Railway, and remained in the employ of that Company, most of the time as Assistant Chief Engineer, until January 1st, 1889.

During the years of active construction by the Santa Fé, Mr. Kingman built, in the years 1886 and 1887, 1 353 miles of railway, somewhat more than 2 miles for each working day—certainly a record which few engineers have exceeded. Early in 1889 Mr. Kingman was appointed City Engineer by the Mayor of the City of Topeka, Kans., and served under three mayors in that capacity.

In May, 1894, Mr. Kingman became connected with the Engineering Department of the Mexican Central Railway Company, under Mr. A. A. Robinson, President of that Company, and, shortly after that was made Chief Engineer, which position he held for nearly twelve years.

When the Mexican Central Railway was taken over by the Mexican Government in 1907 and became a part of the National Railways of Mexico, Mr. Kingman took the position of Engineer of Maintenance of Way, later becoming Office Engineer, as the former position required too active a life. Mr. Kingman retained this office until his death, having been at his desk less than twenty-four hours prior thereto.

Mexico City was Mr. Kingman's headquarters during his connection with Mexican Railways, and he built 994 miles of railway in various parts of Mexico connecting with the Mexican Central System. At the time of the last reorganization of the Santa Fé System he was selected as one of a committee of three to appraise the Atlantic and Pacific Railway.

Mr. Kingman was a man of few words, great energy and determination, and a kind heart; a man of uncompromising honesty, who could be trusted with any responsibility he would accept, with entire confidence that the interests of his employers would be conserved under all circumstances; at the same time, he had a judicial mind, so necessary to every successful engineer.

His assistants and subordinates held him in high esteem for his sterling qualities; he was respected by contractors for his fair dealings and just decisions, and this is no small tribute when one recalls the many millions spent under his control during an unusually active and useful life.

Mr. Kingman was elected a Member of the American Society of Civil Engineers on July 1st, 1885, was a charter member of the American Railway Engineering and Maintenance of Way Association, a member of the National Geographical Society, the American Academy of Political and Social Science, of the Franklin Institute, and of the Masonic Fraternity.

Mr. Kingman is survived by a widow and five children now residing in Topeka, Kans. His oldest son was a student in the Kansas State University at the time of his father's death.

HENRY FIDDEMAN LOFLAND, M. Am. Soc. C. E.*

DIED JANUARY 12TH, 1912.

Henry Fiddeman Lofland was born in Milford, Del., on June 24th, 1862. He was graduated from the University of Virginia in 1883 with the degree of C. E., and immediately became connected with the Baltimore and Ohio Railroad as Assistant Engineer on the construction of the substructure of the Susquehanna River Bridge, near Havre de Grace, Md., on the line then being built between Baltimore and Philadelphia. The building of the substructure of this bridge was notable as being one of the most important operations in deep pneumatic foundation work that had been undertaken up to that time, and on its successful completion, Mr. Lofland served in the same capacity on the construction of the substructure of the Baltimore and Ohio Bridge over the Schuykill River, in Philadelphia.

In June, 1886, on the completion of the latter work, he was engaged as Engineer in charge of bridge substructures on the East Georgia and Florida Railroad, and in December of the same year became Division Engineer on the Mobile and Birmingham Railroad, having charge of building the substructure of the Tombigbee River Bridge. In July, 1887, he was made Division Engineer on the construction of the Louisville, St. Louis, and Texas Railroad, remaining in that position until February, 1888, when he took charge of the surveys for the Louisville and Jeffersonville Bridge over the Ohio River, near Louisville, Ky.

In April, 1888, Mr. Lofland returned to the Baltimore and Ohio Railroad as Assistant Engineer on construction, remaining in this position until April, 1891, when he accepted an appointment as Assistant Engineer of Erection with the Edge Moor Bridge Works, of Wilmington, Del. In 1897, he was promoted to the position of Engineer of Erection, which position he filled until the formation of the American Bridge Company, in 1900, when he was appointed Assistant Engineer of Erection of that Company.

Upon the formation of the American Bridge Company of New York, in 1901, he was made Division Erecting Manager, having charge of all the erection and outside construction work of this Company in the Eastern section of the United States, with headquarters at Philadelphia. He held this position until 1906, when he was promoted to General Manager of Erection, in general charge of all outside construction work, which position he held at the time of his death.

Mr. Lofland's engagement with the Edge Moor Bridge Works in 1891 proved to be the turning point in his career, for from that time

* Memoir prepared by S. P. Mitchell, M. Am. Soc. C. E.

his professional activity was directed exclusively to specializing in the erection of bridges and steel structures; however, the broad knowledge gained from his experience in general railroad construction and the building of substructures, was undoubtedly of the greatest value as a basis on which to found his life work. At the period mentioned, the present-day problems in bridge erection, due to the enormous increase in size and weight of structures, and particularly the problems encountered in the renewal of railway bridges under the most exacting traffic conditions, existed only to a limited degree; consequently, the organization, methods, and appliances in use were naturally crude, and this branch of bridge building was only beginning to be recognized as a specialty requiring the ingenuity and skill of educated engineers. It may be fairly said that no one has contributed more than Mr. Lofland to the marked advancement and development which has taken place in this art in the last twenty years, during which time he was connected with the erection of some of the largest bridges in the United States.

Mr. Lofland was the happy possessor of those qualities making for success which are seldom found in one person—natural engineering talent, resourcefulness, force of character, and remarkable energy, combined with executive ability of a very high order and the faculty of handling men. He was, further, a man of sterling integrity in his business and personal relations, and commanded the respect and admiration of his friends and opponents as well.

In addition to his professional qualifications, there was another side to Mr. Lofland's character which stood out with equal prominence, namely, his social gifts. Possessed of a genial personality, engaging manner, and an inexhaustible fund of natural humor, which made him a delightful companion, he formed lasting friendships wherever he went. He was a raconteur of reputation, not only in the usual sense, but invented many of his own stories, and could turn the most commonplace incident into a humorous anecdote. With his love of social life and his great popularity with both sexes, it is rather remarkable that he never married.

Mr. Lofland's death was not only a severe blow to his friends, but a serious loss to the Engineering Profession. He had been in poor health for three years, but did not give up active work until July 15th, 1911, when he was granted a year's leave of absence, and returned to his old home in Milford, Del., where he died on January 12th, 1912. He is survived by one brother and three sisters.

Mr. Lofland was elected a Member of the American Society of Civil Engineers on June 2d, 1897.

FRANK OTIS MELCHER, M. Am. Soc. C. E.*

DIED JANUARY 22D, 1912.

Frank Otis Melcher was born on June 14th, 1864, at Damariscotta, Me. He was graduated from Tufts College, Massachusetts, in 1887, with the degree of A. M. B. In 1896 he received the degree of Civil Engineer from that college.

After his graduation Mr. Melcher commenced work on the Fitchburg Railroad as Instrumentman, and, later, was appointed Assistant Engineer and then Chief Engineer of that road. In 1897 he was appointed Division Superintendent, and was promoted to General Superintendent during the following year. From July, 1900, to November 1st, 1902, he was Superintendent of the Fitchburg Division of the Boston and Maine Railroad, which had taken over the Fitchburg Railroad.

On the latter date Mr. Melcher became Superintendent of the Illinois Division of the Chicago, Rock Island and Pacific Railroad; in February, 1904, he was made General Superintendent of the Choctaw District, and in June, 1905, General Manager of the Central and Northern Districts. In December, 1906, Mr. Melcher's jurisdiction as General Manager was extended over the entire system of the Chicago, Rock Island and Pacific Railroad, and on the dissolution of the Rock Island-Frisco System, in December, 1910, he was elected Second Vice-President, in charge of the Operating Department of the Rock Island road, succeeding Mr. H. U. Mudge, who was elected President.

Mr. Melcher was a prominent operating officer, and had achieved great and deserved reputation and prestige by his work as Chairman of the Special Committee on the Relations of Railway Operation to Legislation. While he occupied that position he developed a remarkable breadth of view in reference to the regulation of railroads, and great skill in dealing with commissions, lawmakers, and representatives of the railway brotherhoods, and in creating a good understanding and healthy co-operation between them and the railroads at a time when peculiarly needed. His success in these matters was due to the fact that he was a skillful diplomatist, and was also clear-headed and honest.

Mr. Melcher met his death in a railroad accident on January 22d, 1912. With James T. Harahan, Sr., and other railroad officials, he was on his way from Chicago to Memphis, and while his train was standing near Kinmundy, Ill., an express crashed into the rear coach, killing Mr. Harahan, Mr. Melcher, and others.

Frank Otis Melcher was elected a Member of the American Society of Civil Engineers on March 3d, 1897.

* Memoir prepared by the Secretary from information supplied by J. B. Berry, M. Am. Soc. C. E., and from papers on file at the House of the Society.

EDWARD PAYSON NORTH, M. Am. Soc. C. E.*

DIED JULY 20TH, 1911.

The wise men who founded the American Society of Civil Engineers have all passed away.

Their immediate and faithful successors, the men who completed the foundation, and built the superstructure, are in turn fast passing away. The subject of this memoir was one of these.

Edward Payson North was a New Englander by lineage and nativity. He was born at Hartford, Conn., on July 16th, 1835. He died in New York on July 20th, 1911. He was the second son and youngest child of Dr. Milo L. North, a well-known physician, and member of the Class of 1814 at Yale. His mother was Julia Smith, a graduate of Miss Pierce's famous pioneer school at Litchfield, Conn., her father being the Rev. Daniel Smith, for fifty-three years the pastor of the Congregational Church of Stamford, Conn. Her mother was the daughter of Rev. Cotton Mather Smith, who was for more than fifty years pastor of the Congregational Church of Sharon, Conn. Her uncle, John Cotton Smith, was the last Federal Governor of Connecticut, retiring in 1817.

Dr. North's practice was in Hartford, until about 1838, when he moved to Saratoga Springs, N. Y., and there continued actively at work in his profession until his death some eighteen years later.

As a boy, Edward P. North attended a private school in Saratoga, was taught Greek and Latin by his father, and mathematics by his older brother, Thomas M. North, now a well-known member of the New York Bar. Eventually, he was fitted for college at the Canning School, in Stockbridge, Mass. Having determined to enter the Profession of Engineering, he confirmed his choice and obtained a practical foundation for subsequent professional study, by obtaining two years' service in subordinate positions under the late Oliver H. Lee, first, in 1851, as Rodman on the construction of the second track of the Hudson River Railroad, and subsequently, in 1853, on surveys for and the construction of the Chicago, Alton and St. Louis Railroad.

In 1854 he entered Union College as a sophomore of the Class of 1856 in the Department of Science and Engineering.

Socially he became a member of the Sigma Phi Society, and throughout his life maintained his interest and popularity in that venerable fraternity.

Union College, at that time, had obtained a high reputation for its school of engineering, which had been established and built up by the genius, enthusiasm, and liberality of that gifted scholar, the late Professor William Mitchell Gillespie.

* Memoir prepared by Edgar B. Van Winkle, M. Am. Soc. C. E.

Young North had completed all but the last term of his senior year when he received an invitation to join our late distinguished president, Mr. E. S. Chesbrough, as an Assistant Engineer on the great work of constructing a sewerage system for Chicago. This appointment was too attractive to be neglected. Mr. Chesbrough was a leader in his profession, and the work called for was unique and of the greatest educational value from a professional standpoint. No wonder the young engineer promptly embraced the opportunity offered, leaving his course in college to be completed later. Both Mr. Chesbrough and General Webster, the Sewerage Commissioner of Chicago, have spoken in high terms of the intelligence and reliability shown by Mr. North in this work.

During cessation of activity on this work, in the winters of 1857, 1858, and 1859, he returned to Union College to study mining engineering, and, incidentally, took a post-graduate course in chemistry, giving much time to working on blow-pipe analysis in the chemical laboratory under the immediate supervision of Dr. Charles F. Chandler, since Professor of Chemistry in Columbia University and most prominent as an American chemist.

Mr. North had thus the good fortune to have known in his time the venerable Dr. Eliphalet Nott, then still President of Union College, and to have enjoyed the intimate personal instruction of Professor Gillespie and Dr. Chandler, both not only advanced scientific scholars, but instructors having that rare faculty of systematic clear exposition and thorough teaching, combined with such enthusiasm for their respective professions as to infect likewise those studying under them. It is interesting to note that Dr. Chandler had just begun his brilliant career, and young North heard him deliver his first lecture. Fifty-three years later, Dr. Chandler invited his old pupil to attend his last lecture, and during his address humorously alluded to him as "Exhibit A."

It was while at work in the laboratory in the winter of 1858-9 that Mr. North's eyes began to give way. He continued for another season with Mr. Chesbrough, when it became evident that his eyesight could be saved only by complete rest, and living as much as possible in the open air.

This crisis was happily met. He purchased a small, attractively wooded farm on the banks of Rock River, near Sterling, Illinois, to which he retired with his young wife, having just been married to Miss Kate L. Westcott, of Saratoga Springs, the choice friend of his early youth, who survives him.

The anticipated deprivation from reading and study, and the loneliness of a country life, were by this union transmuted into the sweet pleasures of simple home life. He found congenial employment in the open air in cultivating his farm and garden, while his eyes were constantly spared their use, his wife, acting as reader and amanuensis; so

Edward P. North

the dreaded years of separation from his profession and kindred proved in reality to be filled with domestic enjoyment and healthful repose. Some four years later, in 1864, his eyesight having become perfectly restored, he once more took up his profession, and received an appointment as Assistant Engineer on the then called Saratoga and Hudson River Railroad, running from Athens to Schenectady, now a part of the West Shore Railroad.

At about this time Mr. North lost his only child, James Westcott North, a boy of six years. In 1866 he was appointed as Chief Engineer of the Stamford and New Canaan Railroad, now known as the New Canaan Branch, in the great system of the New York, New Haven and Hartford Railroad. This road he located and constructed. The location was so excellent that the line he laid down has never to this day been changed in plan or profile. It was on this work that Mr. North experimented with the use of nitro-glycerine in rock excavation. He must have been one of the first engineers in the country to try it and advocate its use, as its application to blasting had been patented only three or four years before. When we consider its universal present-day use as an explosive, in the shape of dynamite, we can appreciate his far-sightedness as a chemist and engineer. About 1867 he made some preliminary surveys for projected railroads in Iowa, and in 1868 had the good fortune to be connected with the building of that monumental work, the Union Pacific Railway, or the eastern half of the first American transcontinental railway.

General Grenville M. Dodge was the Chief Engineer, Silas Seymour, Consulting Engineer, and Samuel B. Reed, M. Am. Soc. C. E., Chief Engineer of Construction. Mr. North was in charge of the entire work through the Wasatch Mountains, in Utah, some of it of the most difficult and expensive kind, consisting of heavy rock and earth excavation, the protection of the road against the encroachments of a mountain river, tunneling, masonry, bridges, etc., and was very successful in the use of nitro-glycerine in tunnel work. His experience with heavy rock excavation, before dynamite had been invented, was thorough and interesting, and from the data thus obtained, and from previous experience when building the New Canaan Railroad, he subsequently prepared two valuable papers on "Blasting with Nitro-Glycerine"* and "On Blasting Memoranda of Two Blasts Fired April, 1869, on the Union Pacific Railroad."† He assisted in finishing the work in Weber and Echo Cañons, seeing the laying of the last rail and hearing Dr. John Todd's famous prayer on that historic occasion.

In 1872 Mr. North was engaged in making preliminary surveys for a railroad in New Jersey. From 1873 to 1875 he was Principal Assistant to Col. F. V. Farquhar, Corps of Engineers, U. S. A., the

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* *Transactions*, Am. Soc. C. E., Vol. I, p. 13.

† *Transactions*, Am. Soc. C. E., Vol. I, p. 214.

work covering the improvement of the upper Mississippi, between the Falls of St. Anthony and St. Cloud, and the supervision of the harbors of Lakes Michigan and Superior, and of the support of the Falls of Saint Anthony at Minneapolis. In a paper entitled "Wing Dams in the Mississippi above the Falls of St. Anthony,"* Mr. North described some of his work during this period.

In 1876 he was appointed Superintendent of Roads and Streets in the Department of Public Parks of New York City, which department at that time was charged with all public works and their maintenance in the 23d and 24th Wards ("Annexed District"), including all that portion of the city above the Harlem River, some 13 000 acres. This was a suburban district with only one paved street; the other streets and roads were to a degree macadamized, but the large preponderance of them were country roads, merely surfaced with earth.

With meager appropriations, Mr. North made a great improvement in the condition of the streets and roadways. Discarding haphazard primitive methods, he introduced scientific and systematic treatment, and made several practical experiments of much value.

In 1878 Mr. North, in conjunction with John Bogart, M. Am. Soc. C. E., and the late George S. Morison, Past-President, Am. Soc. C. E., prepared an exhibit to represent the American Society of Civil Engineers at the "Exposition Universelle," held in Paris in 1878. This exhibit consisted mostly of designs and photographs of great engineering works recently constructed, or in course of construction, in the United States—notably such as foundations, dams, locks, bridges and viaducts, hydraulic machinery, railroads and their equipment, river and harbor improvements, etc.† So interesting and well prepared was this exhibit that the expert jury reporting on it asked that an exceptional prize be awarded.

Mr. North visited this exhibit after its installation in Paris, passing the summer of 1878 abroad, where he made special study of European roads and their care, particularly as to the use of asphalt for street surfacing in London and Paris. The result of this study was the preparation of a very interesting and exhaustive paper entitled "The Construction and Maintenance of Roads."‡ This treatise was esteemed of such originality and value as to become a standard, and won for him, in 1879, the Norman Medal of the Society in special commendation for its merit as a contribution to engineering science.

In 1880 Mr. North was employed as Consulting Engineer in relation to irrigation for a sugar plantation near Santiago, Cuba, and afterward relative to a water supply for the Ortiz Mine, in New Mexico.

He became, in 1881, Chief Engineer and General Superintendent of the Sinaloa and Durango Railroad, with headquarters at Culiacan, California. He made the first reconnaissance for it, including crossing

* *Transactions*, Am. Soc. C. E., Vol. VI, p. 268.

† *Transactions*, Am. Soc. C. E., Vol. VII, p. 317.

‡ *Transactions*, Am. Soc. C. E., Vol. VIII, p. 95.

the Sierra Madre, at an elevation of 10 000 to 11 000 ft. above sea-level, and superintended construction until the rails were partly laid on the only section ever built, namely, from Altata, on the Gulf of Mexico, to Culiacan, the capital of the State of Sinaloa, some 26 miles. The sanitary conditions of Culiacan were deplorable, and the natives so careless about vaccination, that varioloid was prevalent, and was contracted by Mr. North, forcing him eventually to return to New York, where he became the Vice-President and General Manager of the Railroad Company.

Subsequently he acted as Consulting Engineer for the electric subways in New York, and from time to time was called as an engineering expert to various cities, one of which, in 1906, was Joliet, Ill., to examine the water-power development under way by the Sanitary District of Chicago.

Mr. North visited Europe a number of times. He attended the Fifth International Congress on Inland Navigation, held in Paris in 1892, and presented a report, entitled "Notes on the Relations between Railroads and Waterways in the United States," which had the honor to be printed by the Congress in English, French, and German. He also took a prominent part in the discussion at that Congress. He studied the system of canals in Holland, the control of rivers in France, and, when in Italy attending a session of this Inland Navigation Congress—during his last visit to Europe—at Rome he made a special study of the railways just taken from the contractors by the Government, and prepared an interesting article on Italian railways.* He made many friends among distinguished European engineers and officials, resulting in much kindness and marked courtesies on many occasions, particularly in Rome and when visiting Sicily.

Mr. North, from the reputation established by his paper, "The Construction and Maintenance of Roads," became the leading authority at that time on roadway pavements, and in 1895, under Mayor Strong's administration, was appointed "Water Purveyor," actually the head of that Bureau in the Department of Public Works of New York City, charged with paving and repairing streets. Under his jurisdiction at this time was initiated a general change from stone to asphalt pavements, many miles of the latter being laid and the appearance of the City noticeably improved thereby. A minor work, but one of great practical utility, which he carried out at this time in connection with laying asphalt pavements for the roadways of Park Avenue, was to modify the absurd and dangerous crowning of its pavement, lowering the crown of the roadway as much as 18 in. in some places. While Water Purveyor he read before this Society a paper entitled "The Influence of Rails on Street Pavements,"† not only valuable in itself

* Published in *The Railroad Gazette*.

† *Transactions*, Am. Soc. C. E., Vol. XXXVII, p. 70.

but in the information brought out in the discussion which followed its publication.

Two years later, in 1897, he was made Consulting Engineer of the Department of Public Works of the City of New York, a position of great importance in view of the immense amount and variety of work carried on by that Department, and the new problems in municipal engineering constantly arising.

In 1897 Mr. North was appointed Consulting Engineer to Governor Black's Canal Investigating Commission, and in association with Lyman E. Cooley, M. Am. Soc. C. E., made a report in 1898 embodying certain modifications in the proposed route for the enlargement of the canals of the State of New York, and very full estimates of the cost of construction of the enlarged canal from Lake Erie to the Hudson River.

From a literary standpoint, Mr. North wrote well and clearly on many professional subjects. Whatever he wrote, whether formal papers, journalistic contributions, or technical reports, was characterized by earnestness, thorough study, and sincerity.

In addition to the papers prepared by him for this Society, and for other professional bodies briefly mentioned above, he was at one time a frequent contributor to both *Engineering News* and *The Engineering Record.* He also contributed valuable articles on transportation to *The North American Review* and to *The Forum.*

One of his last papers was prepared for the International Engineering Congress of 1904, on "The Concurrent Development of Traffic on Improved Waterways and on Railroads."*

Numerous additions were made by Mr. North to the discussions following the reading of papers before this Society. His technical education, the broad field of his practical experience, and his great store of well-garnered facts, gleaned from extensive professional reading, was an admirable equipment for taking a prominent, or leading part, in such discussions.

In this connection might be recalled his strength as a debater. Speaking of his earnest advocacy of the tariff and its operations, it was said of Mr. North, by one who knew him well: "Nobody ever entered into a controversy with him on the subject more than once, for his wide range of information, and wonderful memory for statistics, made him an ideal debater."

The great diversity of his interests and knowledge can best be appreciated by glancing at the *Transactions* of this Society and noting his contributions to the various discussions recorded: "Resistances of Railroad Curves," "Nomenclature of Building Stones and of Stone Masonry," "Railroad Construction," "Inter-oceanic Canal Projects," "Water-proof Coverings," "Temperature of Water at Various Depths in Lakes and Oceans," "Enlargement of the Erie Canal," "Preservation

* *Transactions*, Am. Soc. C. E., Vol. LIV, Part B, p. 475.

of Timber," "Preservation of Forests," "South Pass Jetties," "English and American Railroads Compared," "Railroad Levels," "Sewage Disposal," "Mean Horse-Power of a Stream," "Street Railway Track," "Subaqueous Foundations," "Use of Asphaltum in Building Sea Walls," "Construction of Railway Tracks," "Right of Way for Railroads," "The Proposed Lake Erie and Ohio River Canal," "Brick Manufacture and Brick Pavement," "The Holland Dikes," "Controverted Questions in Road Construction," "Electric Rock Blasting," "Inland Transportation," "The Water-Works of Denver, Colorado," "Effect of Depth upon Artificial Waterways," "Construction of a Water System for Placer Mining and Suggestions for a New Method of Dam Building," "Asphalt and Asphalt Pavements," "Lake Front Improvements, Chicago, Ill.," "Theory and Practice of Special Assessments," "Economic Depth for Canals," "Road Building," "Street Grades and Cross-Sections," "Jordan Level, Erie Canal," "Canals from the Lakes to New York," "Railroad Freight Differentials," "The Bohio Dam," "Improvement of Rivers," "Nicaragua Canal," "Preservation of Materials of Construction," "Pavements," and "Forests, Reservoirs, and Stream Flow."

Mr. North might almost be called one of the founders of the American Society of Civil Engineers, although not actually one of those thirteen far-seeing men who in 1852 banded together, not to gain more pay, do less work, or stifle competition by limiting the number of young men entering the Profession, but rather, in union, to broaden its usefulness by increased education and higher ethics, and to increase that mutual respect which inevitably follows better personal acquaintance between men of honor and intellect.

Mr. North was not actually one of these founders, yet he was in fact a founder inspired with all their traditions, for he was one of the earliest to be chosen to reinforce the founders, the majority of whom, far called by the exigencies of war, had been unable to maintain an active organization after 1854.

With a restored Union came the revival of our Society in 1867 and the new inspiration inherent with younger men. Mr. North was one of the first of those chosen at this time, his membership dating from December 4th, 1867. His sponsors were such men as E. S. Chesbrough, General George S. Greene, Alfred Craven, and others.

He was ever an enthusiastic and loyal member, rarely missing any of its meetings, always ready to serve by actual work and counsel to aid in its firm establishment and uplift. Mr. North joined the Society when it had but 26 members. When he died there were not less than 6 000 on its rolls. Our Society has attained grand proportions and prestige. The unselfish men like Edward P. North, who alone for the love of their chosen profession laid its solid foundation, were unconscious that they were preparing for a grand superstructure that would be their lasting monument.

He was made a Director of the Society in 1891, and was chosen its Vice-President for 1898 and 1899. He always attended the Annual Conventions and added not a little to their attractiveness by his genial presence and ability to discuss the papers there presented.

An unusual and dominating factor in the professional career of Mr. North was his love for his profession first, and always for itself, and only secondarily from a business standpoint. This was largely due to his intellectual nature, and that he had become imbued with the enthusiasm of his great teacher, Professor Gillespie, for the theoretical beauties of his profession. With neither master nor pupil would mere remuneration weigh against work intrinsically interesting.

No memoir of Mr. North would be complete without reference to his character and characteristics. He had a pleasing personality, well suited to his social disposition, and naturally gained and kept many friends.

Perhaps his most striking quality, at least the one most appealing to the younger members of the Profession, was his cheerfulness and affability of manner. He was deservedly popular with them, as he was ever ready to give them help and encouragement, not only by word but he would often go much out of his way to aid young engineers by giving, or securing for them appointments. One who was with him as a young assistant in Mexico thus speaks of him:

"He was an extremely hard worker, and expected hard and plenty of work from his assistants, but was very much respected and beloved by all of them. I always retained an affectionate and filial regard for him and maintained with him most delightful relationship up to the time of his death. He always looked upon and spoke of those who worked under him as 'his boys.'"

He was entertaining in conversation, and had the delightful faculty of telling of his wide professional experience and incidental adventures, picturesquely, while yet quite free from boast or egotism.

His words were always so tempered with humor and kindliness that no adversary in debate ever felt wounded by them, even when they were most critical. He had that faculty, often so rare with those ardent in debate, of arming himself with logic and facts alone, and never being tempted to use ridicule or sarcasm.

In his professional work he was most honorable, honest, and independent. Of undoubted integrity, no prospective personal advantage ever warped his judgment or action. His superiors, under whom he worked, all spoke of him as thoroughly reliable and as an accomplished man of excellent ability.

In a word, Edward P. North had intellect, manliness, and gentleness, "That gentleness, which, when it mates with manhood, makes a man."

LA FAYETTE OLNEY, M. Am. Soc. C. E.*

Died March 2d, 1912.

La Fayette Olney, the son of James and Phœbe Smith Olney, was born on June 20th, 1836, at Westmoreland, N. Y., and died on March 2d, 1912, at Mahwah, N. J.

He entered Union College, Schenectady, N. Y., on April 23d, 1857, at the age of twenty-one, taking the scientific course, and was graduated with the degree of A. B.

For many years he was a member of the firm of Wells and Olney, Architects, at Lawrence, Kans., and was also employed extensively in surveys of Western railroads, particularly those now forming part of the 'Frisco and Missouri, Kansas and Texas Railroad systems.

He made extensive surveys in New York City, and was also connected with the New York State Department of Public Works in the office of the State Engineer and Surveyor, having charge of work on New York State canals at Owego, N. Y. Later, he opened an office at 99 Nassau Street, New York City, where he was engaged as Consulting Engineer until his health failed and compelled him to give up active business.

Mr. Olney was a great lover of nature, as well as of the artistic, and had much ability as an artist, doing a great deal of beautiful wood carving, and painting and sketching in oil and water colors. During the later years of his life he copied many of the old masters in the galleries of France and elsewhere. He was a great reader and a fluent French, German, and English scholar. He travelled extensively, crossing the Atlantic fourteen times.

Mr. Olney did much good and charitable work and aided many, both in his own family and outside of it. His was an admirable character, strong, courteous, and courageous. He was a Mason and a Knight Templar. His widow, Elizabeth Hopper Hopkins Olney, formerly of Glen Cove, Long Island, survives him.

Mr. Olney was elected a Member of the American Society of Civil Engineers on October 7th, 1868.

* Memoir prepared by the Secretary from information furnished by Mrs. Olney, and from papers on file at the House of the Society.

ALFRED FRANCIS SEARS, M. Am. Soc. C. E.*

DIED JUNE 7TH, 1911.

Alfred Francis Sears, son of Zebina and Elizabeth Sears, was born in Boston, Mass., on November 10th, 1826. He was educated in the public schools of that city, and was graduated from the Winthrop School with a Franklin Medal in 1841, and from the English High School in 1844. Then, "by a mother's wisdom," as he has said, he was given a year's training in a merchant's counting-room.

In 1845, he entered an architect's office to begin his chosen profession, but, after a year of the close confinement of the drafting-room, the threatening danger of pulmonary disease compelled him to seek out-of-door employment, and he took up civil engineering. He often spoke with pride of having begun civil engineering work "at the head of a hundred-foot chain on the surveys for the Boston Water-Works."

In 1861, when the Civil War came upon the country, Mr. Sears was Engineer of Streets in Newark, N. J. He resigned in June of that year to raise a company, which, being completed in August, was taken to New York City and there incorporated in the First New York Engineers. His company sailed in October with the Expeditionary Corps, referred to at the time as the "The Great E. C." After a year of service in the field, during which, with his company, he assisted in the capture of Forts Walker and Beauregard, in Port Royal Bay, he was ordered to report to the Chief Engineer of the Army, at Washington, and was placed in charge of the construction of Fort Clinch, Fla., where he remained until November, 1865.

After the war he returned to Newark and was made Chief Engineer of what was then known as the Newark and New York Railroad, now a part of the New Jersey Central Railroad. In his autobiography Major Sears refers with special pride to his location of this line, and its final adoption, after bitter opposition on the part of most of the company. This line may be said to have been the first elevated railway in the United States.

In 1867, Major Sears was employed on the location of a transcontinental railway in Costa Rica, and in 1869 became associated with the late Henry Meiggs in its construction. In 1872 he was called by Mr. Meiggs to Peru, where he remained until 1880, when the war with Chili paralyzed public works. He returned to the United States, and, except for occasional employment and trips of short duration in Mexico, Peru, and Europe, and a short temporary residence in New York City, made his home in Portland, Ore., where his only son, Judge Sears, also recently deceased, resided.

* Memoir prepared by John T. Whistler, M. Am. Soc. C. E.

In 1849 he was married to Augusta Bassett, of Bridgewater, Mass., who survives him. She is a very dear old lady and though 87 years of age and nearly blind, feels it to be her duty, as did her husband, to bear her troubles cheerfully and be no burden to any one, either socially or otherwise.

Some time before his death Major Sears wrote:

"Just now I am suffering a congestion of anniversaries, having passed the 83d of my birth, the 30th of my arrival in my Oregon home, and the 60th of my marriage. It is an interesting providence of my life, that, while my wife is so nearly blind she cannot see to read, and I am very deaf, she has excellent hearing and I have perfect eye-sight. So that we are the complement of each other, and together make the one being for which man and wife are intended."

He retained his brilliant and entertaining literary and conversational powers almost to the very end of his life. He was invariably cheerful, and usually had a bright little anecdote, however short, to illustrate some point in his conversation. Unlike so many men of this disposition, he had strong convictions and the courage to stand for them at all times. Nevertheless, he was charitable to any one with whose views he could not agree, only requiring that such contrary views be sincere.

Major Sears was a great reader and an ardent student. He read and spoke with great fluency French, Spanish, and Italian, and even in his ordinary conversation, he used the purest of English and the most delightful style.

On July 22d, 1865, he was called on to address the citizens of Nassau County, Florida, on the subject of "The Reorganization of the State." As the circumstances seemed to require him to respond, he did so, and the address is almost a classic. The following brief quotation from this address illustrates at once his courage and his character:

"Now, if this plain talk hurts any man here who pretends to be loyal, I have only to say that he reminds me of a Jew, professing Christianity, who begged men not to speak harshly of Judas Iscariot because he was a countryman of his."

Major Sears was elected a Member of the American Society of Civil Engineers, on June 2d, 1869.

WALTER HERBERT SEARS, M. Am. Soc. C. E.*

Died October 7th, 1911.

Walter Herbert Sears, the son of Thomas B. and Louisa H. (Churchill) Sears, was born in Plymouth, Mass., on December 8th, 1847. He was graduated from the Massachusetts Institute of Technology in 1868.

He spent the remainder of 1868 and the following year at work on Prospect Park, Brooklyn, N. Y. In 1870 and 1871 he was employed in the office of Mr. John B. Henck, a Civil Engineer of Boston, Mass. From 1872 to 1874, Mr. Sears was engaged, as Chief Engineer, on the construction of water-works for Winchester, Mass., and from 1875 to 1879 he held a similar position at Pawtucket, R. I., where he had charge of the preliminary surveys and construction of the water-works.

In 1880-1881, as Chief Engineer, he constructed a water-works system for Stillwater, Minn., and, in 1882-1883, an extension of the water-works system of Winchester, Mass. In 1883-1884 he was an Assistant Engineer of the American Bell Telephone Company, in charge of placing underground wires in the vicinity of Boston, Mass., and Washington, D. C. As Resident Engineer, he constructed, in 1885-1887, a new water supply for Beverly, Mass., and the following four years were spent as Chief Assistant Engineer of the East Jersey Water Company, at Paterson, N. J.

In 1892 and 1893, Mr. Sears was Chief Assistant Engineer on the additional water supply for Rochester, N. Y. For the succeeding ten years, 1893 to 1903, he was engaged in general engineering practice, his work including the renewal of the water supply systems of Plymouth and Lincoln, Mass., and plans for a new water supply for Grand Rapids, Mich.

In 1903, Mr. Sears was engaged as a Department Engineer under the Commission on Additional Water Supply, appointed by Mayor Low to investigate the Catskill and other water projects for New York City, and was in charge of the Catskill Department. Following this engagement he was Resident Engineer for the Northern New Jersey Flood Commission, with offices at Paterson, N. J.

In 1904, Mr. Sears was appointed Division Engineer of the Croton River Division of the Aqueduct Commission of New York City, and had charge of the work in the vicinity of Katonah, N. Y. From August 1st, 1905, to January 9th, 1906, he was Acting Chief Engineer, and from the latter date to April 1st, 1910, Chief Engineer, in charge of the extensions of the Croton Water Supply. During this period the Cross River Reservoir was completed, and construction on the Croton

* Memoir prepared by Emil Kuichling and Alfred Douglas Flinn, Members, Am. Soc. C. E.

Falls Reservoir was begun and carried nearly to completion. During the latter part of his engagement, Mr. Sears was taken ill and was unable to return to active work. He died at his home at Plymouth, Mass., on October 7th, 1911.

In 1897, Mr. Sears was married to Miss Ella M. Blackman of Plymouth, Mass., who, with three children, survives him.

Mr. Sears' professional work was marked by the great thoroughness with which he studied and treated every problem which came before him. He had the faculty of foreseeing physical difficulties, and when they presented themselves, he was ready with plans to surmount them. This preparedness was the result of his constant observation of the forces of Nature, whereby he became familiar with causes, effects, and processes which remain mysterious to less observant men. Little escaped his quick notice, either in active field operations or in quiet holiday rambles.

Such tastes soon developed in him a keen appreciation of the beauties of natural scenery, and when a Park Commission was created in Plymouth, about fifteen years ago, he was easily induced to become a member, remaining in office until his death. The formation of an attractive park system in his native city was a source of absorbing interest to him, and the beautiful results attained are largely due to the artistic plans which he, as a Commissioner, made without financial compensation. His unselfish civic interest was also displayed by presenting the city with an elaborate report on the improvement of its public water-works, and it is gratifying to note that his plans were duly carried out.

Although firm in his conclusions and duties, Mr. Sears was modest, gentle, and lovable in a high degree, and by these qualities he won the respect and sincere affection of many friends and associates who mourn his loss.

Mr. Sears was elected a Member of the American Society of Civil Engineers on October 5th, 1904.

THOMAS GUILFORD SMITH, M. Am. Soc. C. E.*

DIED FEBRUARY 20TH, 1912.

Thomas Guilford Smith, the son of Pemberton and Margaretta Zell Smith, was born in Philadelphia, Pa., on August 27th, 1839, and received his early education in private and public schools of that city. He was graduated from the Central High School with the degree of A. B., delivering the salutatory address before his class. He then entered the Rensselaer Polytechnic Institute, from which he was graduated as a Civil Engineer in 1861.

Mr. Smith began his professional career as a member of the Engineering Department of the Philadelphia and Reading Railroad. While employed in this capacity, he also did post-graduate work leading to the degree of Master of Arts, which was conferred on him in 1863 by the Central High School of Philadelphia.

In 1865 he was appointed Manager of the Philadelphia Sugar Refinery, which position he held for four years. His capabilities were so widely recognized at this time, and later, that he was frequently employed as Consulting Engineer for a number of railroad enterprises.

In 1872 Mr. Smith was sent to Europe in connection with prospective railway projects, and also served as a Delegate to the International Prison Congress held in London, England, the same year.

In 1873, he was made Secretary of the Union Iron Company of Buffalo, N. Y., which city he afterward made his home. In 1878, Mr. Smith was appointed Western Sales Agent for the Philadelphia and Reading Coal and Iron Company, and was put in charge of the Jersey Shore, Pine Creek and Buffalo Railroad, now the Pennsylvania Division of the New York Central System.

In 1883, Mr. Smith entered into partnership with Mr. J. J. Albright, under the firm name of Albright and Smith, the firm acting as Sales Agent for the Philadelphia and Reading Coal and Iron Company in New York State and Canada.

In 1889, he was appointed Sales Agent for Carnegie, Phipps and Company, Limited. This firm was later merged with the Carnegie Steel Company, and Mr. Smith was made its Buffalo representative, which position he held until he retired from active business in the summer of 1911.

After his retirement Mr. Smith devoted his time to his interests in charitable and educational work, until February 20th, 1912, when he was stricken with a hemorrhage of the brain which caused his death.

Mr. Smith was one of the most prominent and public-spirited

* Memoir prepared by Mr. Henry R. Howland, Superintendent of the Buffalo Society of Natural Sciences, Buffalo, N. Y.

citizens of Buffalo, having been actively identified with important business interests in that city. He occupied a place peculiarly his own in the community, through his love for humanity and his sympathy for and friendly interest in those things which stand for the best in life. It was largely due to his initiative and fostering care that the Charity Organization Society of Buffalo was founded in 1878 after the London model, the first work of the kind in the United States. He served as the active President of this Society for twenty-nine years, and for five years preceding his death he was its Honorary President.

In educational activities, Mr. Smith rendered most useful service to his city and State. At the time of his death he was a Regent of the University of the State of New York, an honorable office which he had held for twenty-two years. He was also President of the Buffalo Society of Natural Sciences, having held that position for nearly ten years.

Mr. Smith was a member of many scientific and literary societies, including the Academy of Natural Sciences of Philadelphia, the Franklin Institute, the American Institute of Mining Engineers, and the Historical Society of Pennsylvania. In 1894, he was made an honorary member of the Phi Beta Kappa Fraternity by the Hobart College Chapter. In 1899 he received the honorary degree of LL.D. from Hobart College, and, in 1903, the same honor was conferred on him by Alfred University, in recognition of his services in establishing the New York School of Clays and Ceramics at that Institution.

On July 14th, 1864, Mr. Smith was married to Miss Mary Stewart Ives, of Lansingburg, N. Y., who, with two sons, Dr. Chauncey Pelton Smith, of Buffalo, N. Y., and Pemberton Smith, Assoc. M. Am. Soc. C. E., of Buenos Aires, Argentine Republic, survives him.

Mr. Smith was a gentleman of rare courtesy of manner. He was always ready to listen with patience and forbearance to the views of others, but was tenacious of his own convictions, and being gifted with sound judgment, his opinions always commanded respect. In an unusual degree, he had the affection of those who knew him.

Mr. Smith was elected a Member of the American Society of Civil Engineers on September 6th, 1871. In 1894 he was appointed a delegate to represent the Society at the Eleventh International Congress of Medicine held in Rome, and he also served as Director from 1894 to 1896.

WILLIAM PARSONS WATSON, M. Am. Soc. C. E.*

Died December 19th, 1910.

William Parsons Watson was born at Sycamore, Tenn., on August 1st, 1850. His father, Judge Samuel Watson, moved from Rhode Island to Nashville, Tenn., where, with the du Ponts, he established the Sycamore Powder Mills. His mother was Miss Charlotte Morton, a daughter of Governor Marcus Morton, of Massachusetts.

Mr. Watson was graduated from Yale College in 1869, and after holding various subordinate positions, was appointed, in 1874, Superintendent of the construction and operation of the du Pont and Company's Powder Works, at Sycamore, Tenn., building head and tail races, erecting masonry structures, machinery, etc. He held this position until 1879, when he went to Washington, D. C., where he had charge, for the contractors, of building the B Street Outlet Sewer. In 1880, he was engaged with the United States Coast and Geodetic Survey, in charge of two river and harbor surveys in Maryland and Virginia.

In 1882, Mr. Watson went West, having been appointed Resident Engineer in charge of heavy construction work in the Missouri Cañon, in Montana, for the Northern Pacific Railroad. In 1884, he went to the Canadian Pacific Railway, as Resident Engineer on location, and afterward on construction, in the Gold Range Mountains in British Columbia. He remained with this Company until 1886, when he was engaged on the Montana Central Railway, as Resident and Principal Assistant Engineer in charge of location and construction, for the Great Northern Railway.

From 1888 to 1890, Mr. Watson was employed by the Union Pacific Railway, in charge of the location of its line in Idaho, Montana, and Washington, serving for six months of this time as Chief Engineer on the preliminary surveys of the Portland and Puget Sound Railroad. In 1890, he was appointed Principal Assistant Engineer in charge of the location and construction of the Seattle and Montana Railway, in Washington, completing the work and turning it over to the Operating Department, in 1892.

In 1892, Mr. Watson was employed by the United States Engineer Commission, to make surveys and estimates for a steamboat railway or canal around The Dalles of the Columbia River, in Oregon, which position he held until 1893 when he was appointed United States Surveyor General of the State of Washington, remaining in that office until 1897.

In 1898, Mr. Watson returned to the Union Pacific Railroad, having charge of reconnaissance, location surveys, and construction.

* Memoir prepared by J. B. Berry, M. Am. Soc. C. E.

serving as Chief Engineer of the Columbia Valley Branch and as Principal Assistant Engineer in charge of the Leamington Cut-Off, in Utah. He remained with the Union Pacific Railroad until 1903, when he became Principal Assistant Engineer of the Missouri Pacific Railway, having supervision of its new location and construction work. In 1904, he had charge of the rectification of grades on the St. Joseph and Grand Island Branch of the Union Pacific Railroad.

In 1905, Mr. Watson was appointed Chief Engineer of reconnaissance and location of about 300 miles of a railway line in Southern Indiana, and, in 1906, he was employed by the Chicago, Milwaukee and St. Paul Railway in charge of mountain location and construction in Montana. In 1908, he went to St. Louis, Mo., and entered the office of the 'Frisco Railway, making investigations and estimates for change of line, reduction of grades, etc. For a year before his death, Mr. Watson was in charge of the investigations for railroad location in the vicinity of Seattle, Wash.

Mr. Watson was competent in all branches of his profession. He was exceedingly painstaking, never neglecting a single item in his investigations, in order that he might give his clients carefully considered conclusions. Being of a cheerful disposition, he was most pleasing in his relations with his associates, readily accepting and investigating any suggestions made by them. If he differed with them, it was always done in a fine spirit. One seldom meets a man so proficient in his work and of such a uniformly fine disposition, so thoroughly loyal, honest, and conscientious.

Mr. Watson was elected a Member of the American Society of Civil Engineers on June 1st, 1887.

GEORGE HOWARD WHITE, M. Am. Soc. C. E.*

Died December 29th, 1911.

George Howard White was born at Grafton, Mass., on June 9th, 1851. After a course in surveying and higher mathematics at the Grafton High School, he, in January, 1870, entered the office of Mr. W. P. Granger, a civil engineer, at Worcester, Mass. He remained with Mr. Granger until April 1st, 1870, when he was appointed Rodman on the surveys for the Adirondack Railroad, being promoted, within two months, to the position of Levelman. He remained on this work until February, 1871.

In the spring of the same year, Mr. White entered the Worcester Polytechnic Institute. After passing all the examinations except those of the senior year, he, in 1872, entered the office of Mr. John Ellis, at Woonsocket, R. I., with whom he remained until January 1st, 1873. He was also employed by Mr. Herbert Keith on surveys of preliminary lines in the vicinity of Boston, Mass.

In April, 1873, Mr. White went West, settling in Minneapolis, Minn., where he was engaged on city and county surveys and on road construction for Mr. Franklin Cook, the owner of some stone quarries along the Mississippi River, in the eastern part of the City. In 1874, he was appointed Assistant to the Chief of Party on the survey of the St. Croix River, from Taylor's Falls to its junction with the Mississippi. After the completion of this work, Mr. White returned to Minneapolis in the employ of Mr. Cook, remaining until March, 1875, when he was appointed Instructor in charge of field practice and mapping, in the Civil Engineering classes at the Worcester Polytechnic Institute. He held this position until the fall of 1876, when he was graduated, as a Civil Engineer, with the Class of 1876 of the Institute.

Mr. White then returned to Minneapolis and, in August, 1877, was appointed Assistant Engineer, under the direction of the Assistant General Superintendent, on the Chicago, Milwaukee and St. Paul Railway, with headquarters at Minneapolis. It was at this time, and under his direction, that the Company's engineering office in Minneapolis was established. Mr. White had charge of the construction of what is known as the "Short Line" between Minneapolis and St. Paul, which was built by the Chicago, Milwaukee and St. Paul, and of the erection of the "Short Line Bridge" over the Mississippi in 1878 and 1879. He was also in charge, for the Railway Company, of the construction of the foundations and masonry of a new elevator, as well as of new shops at South Minneapolis. Later, he was made Engineer in Charge of the Division Office at Minneapolis, and of

* Memoir prepared by the Secretary from information on file at the Society House and from a memoir prepared for the Engineers' Club of Minneapolis, Minn., by its Secretary, J. G. Anderson, Assoc. M. Am. Soc. C. E.

surveys relating to maintenance of way on various Western Divisions of the road.

In January, 1884, Mr. White was appointed Professor of Civil Engineering at the Worcester Polytechnic Institute, and while in this position, organized and built up the Civil Engineering Department. He made a special study of bridge engineering, and was engaged in private practice as a Consulting Bridge Engineer, in addition to his educational work.

In June, 1901, he was made Assistant Superintendent of Sewers at Worcester, Mass., and held this position until January, 1906, when he resigned to return to the Chicago, Milwaukee and St. Paul Railway Company, with headquarters in the Division Engineer's office at Minneapolis, Minn. The Company's "Transcontinental Line" (the Chicago, Milwaukee and Puget Sound Railway), from the Missouri River to the Pacific, had been commenced, and the location and construction of its eastern end was done under the direction of the Division Engineer's Office at Minneapolis.

After the completion of this work, Mr. White made a physical valuation of the Chicago, Milwaukee and St. Paul Railway Company's property in Minnesota for the use of the Minnesota Railroad and Warehouse Commission in the "Rate Hearing Case." He was engaged on this work up to within a few months of his death, which occurred at Minneapolis, on December 29th, 1911.

Mr. White was an earnest student of engineering and was honest and conscientious in all his dealings, his career having been marked by extreme devotion to duty both in public and private life. During his residence in Worcester, Mass., he was prominent in public school affairs, being, for nine years, a member of the School Board; he was also a Deacon of the Free Will Baptist Church. He was one of the charter members of the Engineers' Club of Minneapolis, and was elected an Honorary Member of the same Association on February 8th, 1884.

He is survived by his wife, three sons, three daughters, and five grandchildren.

Mr. White was elected a Member of the American Society of Civil Engineers on May 2d, 1883.

NATHAN JACKSON GIBBS, Assoc. M. Am. Soc. C. E.*

DIED DECEMBER 27TH, 1911.

Nathan Jackson Gibbs was born in Norwich, Conn., on December 26th, 1883, and received his primary education in the grammar schools and Academy of that city.

In 1902 and 1904 he was a student at the Massachusetts Institute of Technology; and from there went to Auburn and Brooklyn, N. Y., in the employ of Frank B. Gilbreth, a general contractor. This position led to that of Assistant Superintendent for the Auburn Hame Company, which he held from December, 1904, to May, 1905, with direct charge of various installations and layouts in the new factory of that company. From May, 1905, to September, 1906, Mr. Gibbs was Leveler, Transitman, and Acting Resident Engineer at Ithaca, N. Y., with the New York, Auburn and Lansing Railroad, in charge of construction work, as well as designing culverts, small bridges, etc., and preparing right-of-way maps.

The lure of the Panama Canal drew him to the Isthmus in September, 1906, where, with the Isthmian Canal Commission, he rapidly acquired larger and larger responsibilities, rising from Levelman to Transitman and Instrumentman. During this service he was in charge of borings and locating part of the Canal Zone boundary lines. He also supervised the building of the large reservoir dam at Porto Bello, later becoming Supervisor of the quarry at that station, and finally Superintendent of the station, with a force of twelve hundred men under him. The stone for the Gatun Dam was taken from this quarry, and nearly all of it under Mr. Gibbs' supervision, for he fired the initial blast there, and continued in charge until the end was so near that at the time of his death this supply station for that great engineering work was practically abandoned.

After more than four years of service, in January, 1911, he returned to Norwich for recuperation from a severe attack of nervous prostration. Later, he was in Colorado, where for a short time he was engaged in irrigation work; but, in response to an invitation from the Tomkins Cove Stone Company, at Tomkins Cove, N. Y., he turned back to quarry work as his specialty. This position was exceedingly attractive to him because of the wholly modern equipment of the plant, one of the largest in the United States; and, as its Superintendent, he quickly won the confidence of the owners and the men under him.

On July 12th, 1911, Mr. Gibbs was married, in New York City, to Miss Emma Grace Wright, of Auburn, N. Y., and Boston, Mass., and

* Memoir prepared by the Reverend Herbert J. Wyckoff.

with her was fast making new friends in Tomkins Cove, where they had lived since August of that year. The accident which caused his death, on December 27th, 1911, was peculiarly unexpected, even in quarry work. A shot, fired at the noon hour, left the face, under which some of the men had been working, in a condition which Mr. Gibbs considered unusually dangerous; and on resuming work in the afternoon he ordered the shovel withdrawn to a safer position, taking his own stand beside it when newly placed in order that he might watch for and give warning of a possible slide. It was while he was thus watching over the safety of his men that he was killed. A mass of rock, close beside and a little above him, and about which there appeared to be no cause for anxiety, fell suddenly, and buried him almost in his tracks. Death was instantaneous, and overtook him alone of all at work in that part of the quarry. He was buried at Norwich, on Sunday, December 31st, 1911.

Mr. Gibbs was held in rare esteem because of his rapidly increasing expert knowledge, his genial cordiality and warm sympathy. He made friends readily, but wisely; and even those whom he supplanted in his rapid rise in rank and place cherished no resentment toward him. In a very real sense the world is the poorer for such an untimely taking away; but in the larger and truer interpretation of the life he shared so freely and joyously, his contribution to the sum total of the world's work and welfare is a permanent enrichment that will outlast even the stone and concrete monuments of his own faithful building.

Mr. Gibbs was a S. A. E. man of the Massachusetts Institute of Technology; a charter member of the American Society of Engineering Contractors; a member of the Connecticut Society of Civil Engineers, and of the American Railway Engineering Association. He was also a member of the Advisory Board of the Young Men's Christian Association of the Canal Zone, a member of the Strangers Club at Colon, and an honorary member of the Chelsea Boat Club of Norwich.

Mr. Gibbs was elected an Associate Member of the American Society of Civil Engineers on May 2d, 1911.

JOSEPH CANBY HADSALL, Assoc. M. Am. Soc. C. E.*

Died June 29th, 1911.

Joseph Canby Hadsall, the son of Mollie J. (Hunt) and John Edward Hadsall, was born in Pleasant Valley, W. Va., on April 20th, 1873. He received his early education in the schools at Bethany, W. Va., and in 1892 was graduated from the Normal School at West Liberty, W. Va. For two years, 1898-1900, he studied architecture and civil engineering at Columbian University, Washington, D. C.

Mr. Hadsall was employed as Assistant on city work with J. F. Burley and Brother, at Moundsville, W. Va. This was followed by office and field work with the South Penn Oil Company, and, later, by one year of private practice at Moundsville, W. Va., on municipal work and land surveys.

From 1898 to 1905 he was employed in the United States Treasury Department, and for several months in the latter year, in the United States Surveyor General's Office at Cheyenne, Wyo.

From 1905 to 1911, as Civil and Irrigation Engineer for the Wyoming Development Company and the Wheatland Industrial Company, he was responsible for the designs for an irrigation system for 90 000 acres of land, and had charge of all the engineering work. He also designed and constructed the water-works, sewerage, and electric light systems of Wheatland, Wyo.

In June, 1902, he was married to Miss Frances Luttrell, of Knoxville, Tenn., who with a son, Joseph Vernon, survives him.

Mr. Hadsall was a man of sterling character and strict integrity. He won the esteem and respect of all who knew him, and always stood for the highest and best interests of the community. He was a member of the Protestant Episcopal Church.

Mr. Hadsall was elected an Associate Member of the American Society of Civil Engineers on November 8th, 1909.

* Memoir prepared by H. B. Patten, M. Am. Soc. C. E.

WALTER SCOTT HANNA, Assoc. M. Am. Soc. C. E.*

Died July 4th, 1912.

Walter Scott Hanna was born in Lykens, Pa., on September 16th, 1879. He was educated at Pennsylvania State College and Lehigh University, from which latter institution he was graduated in 1902 with the degree of C. E.

During the summer vacations of 1898, 1899, and 1900, Mr. Hanna was employed as Chainman, Leveler, and Transitman on mining engineering corps of subsidiary companies of the Susquehanna Coal Company, at Shamokin and Lykens, Pa., which work proved of immense value to him in his subsequent work at the Massachusetts Institute of Technology, in lecturing on practical mine surveying and rock excavation.

From June to September, 1901, he was engaged as Chief of Party and Assistant to the Maintenance-of-Way Engineer of the Norfolk Division of the Southern Railway, devoting all his spare time during this period to the study of the operation of the Division. In June and July, 1902, Mr. Hanna was Assistant Instructor in Topographical Surveying at the Lehigh University Summer School, and had charge of several field parties. Later, from July to September, he was employed as Assistant to the Superintendent of Blast Furnaces of the Pennsylvania Steel Company, at Steelton, Pa., studying blast-furnace operations and steel manufacture.

In September, 1902, he was appointed Assistant Instructor in Civil Engineering at the Massachusetts Institute of Technology. While in this position, besides being engaged in outside work, he did post-graduate work in Civil Engineering at the Institute.

In June, 1903, Mr. Hanna was employed as Assistant Engineer with the American Pipe Manufacturing Company, of Philadelphia, Pa. While with this Company, he was appointed Resident Engineer of the reservoir, etc., then being built at Scarsdale, N. Y., and also assisted the Chief Engineer with the mathematical calculations for a new water meter.

During August and September, 1904, Mr. Hanna located a railroad at Lykens, Pa., for the Lykens Valley Coal Company. He then removed to Columbus, Ohio, where he was employed as Draftsman for the Board of Public Service, on designs for a sewage pumping station, clear-water reservoir for a filter plant, etc. He remained in this position until January, 1905, when he was engaged as Engineer by Hugh MacRae and Company, Bankers, of Wilmington, N. C., on a land development project.

* Memoir prepared by the Secretary from information furnished by A F. Hanna, Esq., and from papers on file at the House of the Society.

In November, 1905, Mr. Hanna returned to the Massachusetts Institute of Technology as a student and as Assistant Instructor in Civil Engineering, remaining until March, 1906, when he was appointed Assistant Engineer with the State Water Supply Commission of Pennsylvania, being engaged in investigating and making reports on water supply companies, projects, etc. He was also employed on work for the Pennsylvania State Board of Health.

In October, 1910, he was appointed Consulting Engineer with the J. B. Hogg Engineering Company, of Pittsburgh, Pa., but in January, 1911, he was compelled to retire, on account of ill-health. From that time until his death, on July 4th, 1912, Mr. Hanna made his home at Asheville, N. C. He is survived by his widow and one daughter.

Mr. Hanna was elected a Junior of the American Society of Civil Engineers on October 6th, 1903, and an Associate Member on June 5th, 1907.

THOMAS WILLIAM ROSTAD TEIGEN, Assoc. M. Am. Soc. C. E.*

Died September 20th, 1911.

Thomas William Rostad Teigen was born in Manitowoc, Wis., on October 6th, 1881. He received his education in the Grammar School and the Mechanics Arts High School of St. Paul, Minn., and at the University of Minnesota.

From April, 1900, to April, 1902, Mr. Teigen served as Rodman, Topographer and Instrumentman on the Great Northern Railway, on the location and construction of the Jennings Branch in Montana and Canada. From April, 1902, to January, 1903, he was in charge of a party on land surveys in Minnesota and Dakota.

In January, 1903, he entered the employ of the Chicago, Rock Island and Pacific Railway (Choctaw District), serving as Instrumentman, Assistant Engineer, Resident Engineer, and Assistant Division Engineer on Maintenance of Way and Construction, until April, 1906, when he was appointed Assistant Engineer with J. G. White and Company on the construction of the Philippine Railway, in charge of the terminals, wharves, etc., at Cebu.

From April to October, 1907, Mr. Teigen was Engineer of Construction of the Twin City and Lake Superior Railroad Company. While with this company he had charge of the construction of 30 miles of electric road, designed the preliminary plans for a bridge over the St. Croix River, and made standard plans for trestles, culverts, pipes, etc.

In October, 1907, he was appointed Assistant Chief Engineer of the Minneapolis and Rainy River Railway, and made revisions, constructed a 10-mile extension, and made a report and maps for the Interstate Commerce Commission.

In April, 1908, Mr. Teigen went to South America and was put in charge of a party on preliminary surveys and location for the Madeira-Mamoré Railway in Brazil, afterward being appointed Assistant Engineer on Construction with headquarters at Santo Antonio do Rio Madeira. While engaged on this work, he contracted beri-beri, which compelled him to return to the United States in December, 1908.

After a serious operation and a prolonged stay in the hospital at St. Paul, Minn., Mr. Teigen went to the El Paso Southwestern Railway, at El Paso, Tex., as Chief Draftsman and Designer. He designed the plans for the Pintado and French River Bridges and for the Bonita water supply.

Preferring field work, he entered the service of the Mexico Northwestern Railway, in charge of a location party on the extension of this

* Memoir prepared by C. S. McKinney, Esq.

road from Terrazas to Madera, in Chihuahua. On the completion of this location, Mr. Teigen was appointed Office Engineer on Construction at Nueva Cases Grandes, in which capacity he designed the terminal buildings and yards at both Pearson and Madera.

When the construction of the extensive terminals at Madera was begun, Mr. Teigen was appointed Engineer in Charge of Terminals, and had supervision of all the work in connection with this modern and up-to-date plant. The work was progressing satisfactorily when alarming symptoms of his old malady caused the local physicians to order Mr. Teigen to go to St. Paul, Minn., but the disease was too far advanced for an operation to be of any benefit, and, after lingering a few days, he died.

Mr. Teigen was elected a Junior of the American Society of Civil Engineers on September 5th, 1905, and an Associate Member on December 1st, 1908.

WILLIS TUBBS TURNER, Assoc. M. Am. Soc. C. E.*

Died September 7th, 1911.

Willis Tubbs Turner was born at Vineland, N. J., on April 23d, 1869, and was educated at the Grammar and High Schools of that place. In 1891 he began his professional career, first, as Field Assistant for the United States Geological Survey in the Sierra Nevadas, later, serving as Topographer, surveying and mapping various quadrangles in California and Washington, among which the San Antonio, Rock Creek, Tejon, Mt. Pinas, and Santa Ynez maps bear his name.

From 1896 to 1898 he was U. S. Surveyor in Indian Territory, in the subdivision and allotment of the Choctaw Nation lands. This was a rare instance in which the Government made its own land survey, with the interesting result that the cost was about two-thirds of that under the usual contract system, that all errors were corrected, corners were actually monumented, and, included in this cost and incidental to the work, was complete topographic mapping—a result due in great measure to the enthusiasm and sturdy perseverance of such young men as Mr. Turner. He remained on this duty in spite of malaria and typhoid fever, both of which he contracted at various times, but overcame with his robust constitution.

This experience and the mountain training acquired in the field work of the U. S. Geological Survey undoubtedly contributed greatly to his success; as an expert packer and a fine mountaineer, he could lead in exploration and organize his forces to the best advantage.

In June, 1903, Mr. Turner entered the U. S. Reclamation Service, and was in charge of surveys, reservoir sites, and canals on the Malheur, Harney, and Umatilla Projects, in Oregon; the early surveys of the Roosevelt Dam, in Arizona; of the border of Utah Lake and the Jordan River, in Utah; and had charge of the tests for hydraulic-fill dams and canals of the Sun River, in Montana.

In 1906, Mr. Turner was appointed Assistant Engineer of Montana, and was in charge of irrigation investigations, which included a report on the Cherry Creek Project.

In September, 1906, he accepted an offer of the Peruvian Government to take charge of its irrigation investigations, with the title of Chief of the Hydrographic Commission. In this new and important field, which included the whole subject of hydrography, duty of water, and hydro-electric development, he reported on the Lagunes Huarochiri, a series of some 65 lakes, with estimates for storage and plans for hydraulic-fill dams. These lakes are on the head-waters of

* Memoir prepared by William S. Post, Assoc. M. Am. Soc. C. E.

the Rimac River, from which Lima derives its water supply. He also studied the Imperial Pampa, near the Canete River. During this period he was the author of the following publications: "Informas sobre el Rio Chillon," "Las Lagunas Huarochiri y Su Futuro Ensanche," and "Pantana Gallinaza y Su Desague."

Mr. Turner's correspondence at this time is of the greatest interest, the following being quoted in regard to one of his discoveries: "The most remarkable dam site I have ever seen is located some twenty miles above Chosica on the Santa Eulalia River. The height is 460 feet, and 400 feet above stream bed has no greater width than 35 feet."

His letters also contain delightful comments on the methods and customs, which all engineers who have had to do with Central or South American matters, would keenly appreciate. In a characteristic and almost brusque judgment of methods, he refers to "the mania for reports" before facts are obtained.

Mr. Turner returned to the United States in May, 1910, and feeling that he needed a complete change, proposed to take up scientific farming for a year or two and then return to his Profession. He had purchased a farm and was so engaged when he contracted a severe cold which ultimately proved fatal. By the advice of his physician he went to New Mexico in hope of improvement, accompanied by Mrs. Turner. A complication of tubercular meningitis set in, however, and he died at Albuquerque, on September 7th, 1911. His wife and daughter, who frequently accompanied him on his many travels, and who were with him in South America, survive him.

Mr. Turner was a fine type of the Western engineer trained in the school of experience. He was uncompromisingly honest in all his relations and thoroughly understood himself. He was a good judge of human nature, positive, and forceful, and had a subtly humorous manner. Ambitious to excel in all that he undertook, his determination and constitution ensured its completion. His name is on the long list of those who have carried American engineering methods into far countries, and have paid the toll in health and with their lives.

Mr. Turner was elected an Associate Member of the American Society of Civil Engineers on December 7th, 1904.

GEORGE SHREVE WILKINS, Assoc. M. Am. Soc. C. E.*

Died August 17th, 1910.

George Shreve Wilkins was born in Philadelphia, Pa., on March 14th, 1868. Soon after his birth his family moved to Mount Holly, N. J., where he lived until 1882, when he went to school at Lawrenceville, N. J. In 1886 he entered Princeton College, and was graduated in 1890.

He spent two years at Joplin, Mo., as a Mining Engineer, and then returned to Princeton as an instructor in the college. During 1895 and 1896 he was a student in the "Ecole de Mines," Paris, from which he was graduated. In 1899 Mr. Wilkins was Professor of Engineering in the University of Alabama, and held this position while engaged at the Paris Exposition of 1900. The decoration of the Legion of Honor was conferred on him by the French Government, he being the youngest man who had ever been thus honored.

In 1902 Mr. Wilkins returned to the University of Alabama, and shortly after that took a position with the Philippine Exhibition. From 1906 until his death, on August 17th, 1910, he was in the service of New York City as Assistant Engineer in the Bureau of Highways, Borough of the Bronx.

George Shreve Wilkins was elected a Junior of the American Society of Civil Engineers on October 4th, 1892, and an Associate Member on October 5th, 1898.

* Memoir prepared by S. C. Thompson, M. Am. Soc. C. E.

JOSEPH HECKART FRAZER, Jun. Am. Soc. C. E.*

Died August 16th, 1911.

Joseph Heckart Frazer, the son of Eben B. Frazer, was born at Port Deposit, Md., on September 30th, 1882. He was graduated from Delaware College, Newark, Del., in 1903, with the degree of B. C. E. Immediately after his graduation, he entered the service of the Baltimore and Ohio Railroad Company, as Topographer, being engaged on engineering work in Maryland, Pennsylvania, and West Virginia, until November, 1904, when he resigned.

In January, 1905, Mr. Frazer began work with the Bolivian Railway Company, as Transitman on the preliminary surveys of the Viacha-Oruro Line. He was also employed by the Bolivian Government as Engineer with the Commission for the Study of Railways.

In June, 1905, he was appointed Chief Engineer and Assistant Manager of the Concordia Mine, of the Andes Tin Company, being engaged in laying out and building roads and in the erection of an electrical transmission plant. In March, 1907, he resigned this position to form, with Mr. William R. Rumbold, of Tunbridge Wells, England, the firm of Rumbold and Frazer, with headquarters at Oruro, Bolivia. This firm made surveys and estimated costs for a wagon road across the Andes from Caluyo to Concordia Mine; reported on and estimated costs for a water supply for the City of Oruro; examined and reported on mining properties, etc.

In 1909, Mr. Frazer sold his interest in the firm of Rumbold and Frazer to Mr. A. Basil Reece, and became associated with Mr. Adam W. Yount, in the contracting and engineering firm of Yount and Frazer, at Oruro, Bolivia. This firm successfully completed contracts for the construction of important railways in Bolivia, among them being large portions of the lines from Rio Mulato to Potosi, and from Oruro to Cochabamba.

Mr. Frazer was one of the few Americans who have successfully managed large and important contracts with native labor, and, although a young man, had, by hard work and careful judgment, amassed a considerable fortune. He died of pneumonia, at La Paz, Bolivia, on August 16th, 1911, after an illness of seven days.

He was one of the most popular foreigners in Bolivia, a man of high ideals and lovable character, and leaves a record of clean, honest work.

Mr. Frazer was elected a Junior of the American Society of Civil Engineers on March 31st, 1908.

* Memoir prepared by Mr. P. A. Seibert, Mgr., Andes Tin Company, La Paz, Bolivia.

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