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AND
AMERICAN RAILWAY JOURNAL.

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THE CHICAGO SEWER DISTRICT, lying between 79th, 106th and Halsted Sts. and the lake, known as the 95th St. low-level district, has recently been a subject of controversy. This district was shown on the map in our issue of April 21, and, as described in the article on the intercepting sewer system, it has an area of about 10,800 acres. It was proposed to put in about 122,187 ft. of brick sewer, 2½ to 13½ ft. in diameter, leading to a pumping station on 95th St., from which the sewage would be raised and discharged through an outfall into the Calumet River. This river is already a foul stream which pollutes the water of the lake, and it is urged that the pollution should be checked instead of being increased by the sewage from the 95th St. district. Two plans for the latter purpose have been suggested: (1) to connect the sewer system of the district with the new intercepting sewer system; (2) to connect it with the Drainage Canal by means of a channel striking the canal at "the Sag." The latter plan is the one generally favored, and both the Mayor and the Commissioner of Public Works are opposed to any further discharge of sewage into the lake. After a public hearing, at which the property-owners sustained the position of the authorities, the engineers of the sewer department were directed to prepare new plans, providing for carrying the sewage to the Drainage Canal.

A CALUMET RIVER DRAINAGE CANAL is now being planned by the direction of Mr. L. E. McGann, Commissioner of Public Works, of Chicago. The river now flows into the lake, but as it carries a great quantity of sewage and filth from packing houses, etc., it is a source of serious pollution to the city's water supply from the southern intakes. The proposition to discharge the sewage of the 95th St. sewer district into the river has called attention to the present state of affairs, and measures are to be taken to treat the Calumet River like the Chicago River, and reverse its flow, giving a flow westward from the lake into the Chicago Drainage Canal. The general plan is to build a canal about 100 ft. wide and 14 ft. deep, from Lake Calumet along the course of the Little Calumet River to the point on the Drainage Canal known as "the Sag," a distance of about 15 miles. This would have a capacity of 300,000 cu. ft. per minute, and would serve as a feeder for the Drainage Canal. The cost is estimated at $3,000,000. Should there be any objection to this plan, the Calumet canal could be connected with the Illinois & Michigan Canal. A special drainage district may be organized, with a Board of Trustees to carry out the work, but the Commissioner of Public Works hopes that the U. S. Government will take the matter up and build a navigable canal from the mouth of the Calumet River to the Drainage Canal, thus giving a route by which vessels can avoid the detour through the Chicago River. The general plans for the new canal are to be prepared by Mr. John Ericson, City Engineer; Mr. F. E. Davidson, Superintendent of Sewers, and Mr. Wm. Lowe and Mr. Hill, Engineers of the Sewer Department.

THE CAMBRIA STEEL CO., a new corporation, is about to absorb the Cambria Iron Co., of Johnstown, Pa. The new company is to have a capital of $24,000,000, and the Iron Co. is guaranteed 4% dividends and the stockholders may subscribe for three shares in the new company for each of the Iron Co.'s shares held. The Cambria Iron Co. has been in existence nearly 50 years and has a capital of $8,000,000, with assets of $29,000,000 at current low valuations. The works have outgrown this capital and the new company has been formed.

AMERICAN STEEL FOR BELFAST SHIPYARDS is an import that is causing concern among Scottish steel makers, says "The Engineer." The firm of Harland & Wolff, of Belfast, is credited with a consignment of 800 tons of large steel ship-plates from Pittsburg. These plates are 28 × 5 ft. and weigh 2 tons each, and are supposed to be landed at Belfast from $2.40 to $2.88 per ton less than if bought from Scotch or British makers. This Belfast firm consumes in shipbuilding about 40,000 tons of steel and 8,500 tons of iron per year, besides using in this time about 60,000 tons of other materials. The existence of a fine line of steamers running between Belfast and the United States, and the advantages of the rapid delivery of large quantities of steel at low prices, are supposed to account for the demand upon American steel manufacturers. In years past almost the whole of the steel used in the Belfast yards came from the West of Scotland; increased consumption calls for additional supplies from the East Coast, Wales, Barrow and America.

THE RUSSIAN BATTLESHIP which the Cramps, of Philadelphia, are to build will have a displacement of 12,700 tons, and will be able to maintain a speed of 18 knots for 12 consecutive hours, with all coal, ammunition and stores aboard. She is to be superior to any class of battleship now afloat; and the new United States battleships should be her equals, at least. If our builders can furnish other nations with the best vessels of this class in existence, it would be worse than folly to talk of providing inferior warships for our own navy.

THE CONDITION OF ADMIRAL CERVERA'S SHIPS is reported upon as follows by the Chapman-Merritt Wrecking Co.: The "Vizcaya" and "Almirante Oquendo" are total wrecks, not worth saving for junk. The "Maria Teresa" has a hole 15 ft. long in the bottom close to the keel; this is being patched up and she will be first taken into Santiago harbor, and then possibly to Havana to be further fixed up for her journey north. The "Christobal Colon," says Capt. Smith, of the wrecking company, is in such a precarious position that he has little hope of saving her. The "Reina Mercedes," in Santiago harbor, can be easily floated.

THE EIGHT-TRACK DRAWBRIDGE over the Chicago Drainage Canal will be a Scherzer rolling-lift bridge, the contract having been awarded to the Scherzer Rolling Lift Bridge Co., of Chicago, Ill., at its bid of $369,140, exclusive of the operating machinery. The sum of the bid was made up as follows: Plans, $27,000; substructure, $166,575; superstructure, $175,565. The list of bids was given in the supplement of an issue of Aug. 4, and we expect to describe the bridge in a later issue.

A SIX-TRACK DRAWBRIDGE, on the Scherzer rolling-lift system, is to be built for the New York, New Haven & Hartford R. R., to carry the tracks of the new terminal station at Boston, Mass., across the Fort Pond channel. The plans were made by the Scherzer Rolling Lift Bridge Co., of Chicago, and the contract for construction has been let to the Pennsylvania Steel Co.

THE MOST SERIOUS RAILWAY ACCIDENT of the week occurred Aug. 21, on the Providence Division of the New York, New Haven & Hartford R. R., at Sharon, Mass. The accident was a rear-end collision between two passenger trains, and according to reports resulted from the failure of a signal light to work properly. Four persons were killed outright, one seriously injured and 12 or 15 were more or less seriously injured.

A LOCOMOTIVE BOILER EXPLOSION occurred on Aug. 17 in the round house of the Santa Fe, Prescott & Phoenix R. R., at Prescott, Ariz., resulting in the death of two and serious injury of two others. The report states that the firebox and part of the boiler, weighing four or five tons, was blown about 1,200 ft.

THE FALLING OF A PORTION OF THE SIDEWALL of the Carnegie Tunnel on the Chartiers division of the Pittsburg, Cincinnati, Chicago & St. Louis Ry., at Pittsburg, Pa., on Aug. 22, killed 8 men, seriously injured 7 others, two of whom will probably die. The men were part of a force engaged in tearing out the tunnel and were preparing to pull down the west wall when, without warning, the wall fell forward, burying those standing in front.

THE LONDON WATERLOO AND CITY ELECTRIC Underground Railway is now formally opened to traffic, from the Waterloo railway station to the Mansion House, a distance of 1½ miles. This distance is run in five minutes for a fare of 4 cts., or 6 cts. for a round trip. The cars have a central passageway, and they are equipped with Westinghouse air-brakes and are lighted by electricity. An electric third-rail system permits a maximum speed of 25 miles per hour.

AN AMERICAN LOCOMOTIVE FOR ENGLAND, recently built by the Baldwin Locomotive Works, Philadelphia, Pa., is a narrow gage tank engine for the Lynton & Barnstaple Ry., having a gage of 23½ inches. The engine has side tanks and a rear coal bunker behind the enclosed American cab. It has outside cylinders 10 × 16 ins., driving the rear pair of coupled axles with wheels 33 ins. diameter, and under each end of the engine is a pony truck with 22-in. wheels. The weight is 46,000 lbs., of which 29,000 lbs. are on the driving wheels.

THE AMERICAN MAIL STEAMSHIP CO., launched, on Aug. 18, the "Admiral Dewey," the first of the four steamships being built for this company by the Cramps, of Philadelphia. Each of these steamers will be 280 ft. long, 36 ft. beam and 25 ft. depth of hold, the tonnage is 2,000, and a speed of 15 knots is specified. They are twin-screw steamships with triple-expansion engines. The "Admiral Dewey's" keel was laid on May 25, and her hull has consequently been built in about 11 weeks. The contract called for her delivery, complete in all parts, during the month of October.

A JAPANESE SHIP-CANAL is under consideration. It would connect Osaka Bay, on the Pacific side, with the Sea of Japan, by way of Lake Biwa. This lake covers 46% of the length of the line. Including the lake, the length of the canal would be about 75 miles. The estimated cost ranges from $70,000,000 to $150,000,000, and the time of execution from 10 to 15 years.

ONLY ONE BIDDER FOR A NEW WATER SUPPLY for Jersey City responded to the call for bids on Aug. 18. Mr. Patrick H. Flynn, of Brooklyn, submitted two formal propositions, each for water from the Rockaway River, delivered by gravity at an elevation of 210 ft. above mean high tide. The prices under each plan for a plant capable of delivering 50,000,000 gallons a day were about $7,500,000, or about $8,000,000 for a 70,000,000-gallon plant. If the plant should not be purchased until some years after completion the price would be considerably higher. The prices for water by the million gallons ranged from $36 to $20, for quantities varying from 25,000,000 to 70,000,000 gallons per day. Mr. Flynn was one of four bidders last February, and his bid is higher now than then.

BIDS FOR 16 TORPEDO-BOAT DESTROYERS and 12 torpedo boats were opened at the Navy Department, Washington, D. C., on Aug. 23. The specifications were that the destroyers should be of about 400 tons, and should cost not to exceed $295,000 each, with a speed of 28 knots; while the torpedo boats were limited to 150 tons and $170,000 each, with a speed of 26 knots, making a total of about $6,900,000, the amount appropriated for this work. The bids received were in most cases for one, two or three boats, and all were within the figure set by Congress. As a large number of bids were received and as estimates were asked on two propositions, one as planned by the department, the other based on the bidder's idea, it will require some time to sort and tabulate the bids received, which will be necessary before awards can be made. One firm offered to build destroyers with a speed of 40 knots. Neither Cramps nor Herreshoffs presented bids.

RULES FOR APPOINTMENT to the Corps of Civil Engineers, U. S. N., have been issued by Commodore Endicott, Chief of the Bureau of Yards and Docks. The applicants must be 25 years and not more than 35 years old. A naval medical board will examine them as to physical fitness, and a board of officers appointed by the Secretary of the Navy will examine applicants as to their mental and professional qualifications. Each applicant must give evidence of American citizenship and apply to the Secretary of the Navy for a permit to be examined. As to professional qualifications, there must be evidence of having received a degree of civil engineering from some professional institution of good repute, and a record of at least three years' practical experience as a civil engineer. The mental and professional examination will be competitive and in writing. It will comprise the following subjects: Testimonials; English grammar and composition; elementary physics; elementary geology; analytical geometry; differential and integral calculus; applied mathematics, including mechanics of solids and fluids and strains in structures; construction materials; engineering construction; topographical, trigonometrical and hydrographical surveying, and mapping; instruments and their use and adjustment. To pass a satisfactory examination candidates must attain an average of at least 80% in applied mathematics, construction materials and engineering construction, and a general average of 75%. An examination will be held on Sept. 6 at the Navy Yard, Washington, D. C., at 10 a. m.

AN OBSERVATORY ON PIKE'S PEAK is to be built by the Manitou & Pike's Peak Cog Ry. Co., and the contract has been signed for the construction of the tower. It is proposed to mount in this tower four powerful telescopes for the benefit of visitors—and the profit of the railway company.

THE 1,500-HP. ENGINES FOR THE BERLIN, GERMANY, ELECTRIC WORKS.

We illustrate in the accompanying cuts the general details of the two new engines built by the Goerlitz Machine Works, of Goerlitz, Germany, for the Berlin Electrical Works, the company operating the five central lighting stations of Berlin, Germany. These two engines are exactly alike, and, as will be seen from the general view, Fig. 1, they are cross-compound engines with the cylinders supported on vertical cast-iron A-frames. The cranks are set 108° apart and the hollow shaft is 15.75 ins. in diameter at the bearings. For convenience and to avoid the possibility of a complete shut down, this crank shaft is made in two sections, the adjoining ends of which are provided with solid flanges which are bolted together. There are four ample bearings, one on each side of each crank.

The high-pressure cylinder is 36.61 ins. in diameter, the low pressure 57.09 ins. and the stroke is 43.31 ins. With a steam pressure of 10 atmospheres (about 150 lbs.) and a speed of from 105 to 110 revolutions per minute the nominal horsepower is 1,500, although the engine can be worked up to 1,900 HP. Both cylinders are jacketed with live steam.

The steam for operating the engine passes from the boilers through the main steam pipe and valve into the high pressure receiver HR (Fig. 3). From the receiver it enters the high pressure cylinder at the proper time and exhausts into the low presure receiver LR, from which it in turn passes into the low pressure cylinder and thence through the exhaust pipe to the condenser.

The valve mechanism, known as the "Collmann," after its inventor, is operated by two sets of bevel-gears just outside the bearings G G, and two vertical rods which run up alongside the cylinders and drive continuously through spiral gears and a short horizontal shaft. This shaft is provided with eccentrics and bell cranks, which operate the rods controlling the 8 poppet valves, four of which, a, b, c, d, are shown in Fig. 2. The governor is placed on an extension of the high pressure driving rod, Fig. 3. It is of the usual fly-ball type with dash pot and a system of links and levers which adjusts the admission for the high pressure cylinder. The exhaust of the high pressure and valves of the low pressure cylinder are fixed and do not vary with the load.

The crossheads are keyed to the piston rods and are provided with a slide plate which bears against a flat slide guide on one side only of the A-frame. The connecting rod is of the ordinary marine Y type and straddles the crosshead.

The air pumps are operated by levers supported by offsets bolted to the A-frames and connected to the crossheads by suitable links. The diameter of the air pump cylinders is 32.48 ins. and the stroke is 9.45 ins. The jet condenser is fitted with main and auxiliary injectors, the latter being supplied from a cold water tank, and is used only when starting up to avoid disturbing the condensers of other engines which feed from the same water main. The main injector is opened and the auxiliary closed when the engine is running smoothly.

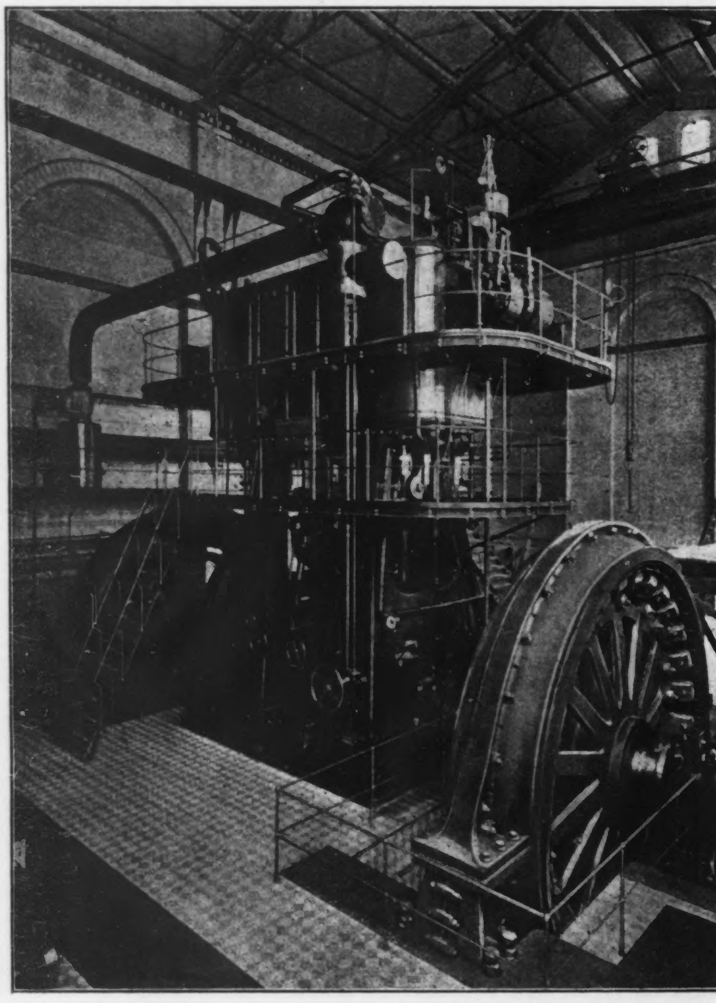


FIG. 1.—A 1,500-HP. CROSS COMPOUND CONDENSING ENGINE FOR THE BERLIN, GERMANY, ELECTRIC WORKS.
The Goerlitz Machine Works, Goerlitz, Germany, Builders.

At each end of the engine and mounted directly upon the shaft is the field of a 700 K-W. 3,000-volt alternating generator. These fields are heavy and serve as balance wheels to steady the engine. The specifications required that with a sudden change of load of 25% the speed should not vary more than 1½%, and that the variation should not be greater than 3% when the full load was thrown off or on. As will be seen from Fig. 1, the various operating valves are conveniently placed at the front within easy reach of the engineer. Lubrication is accomplished from a central point, the "Ritter" system being employed. We are indebted to the "Zietschrift des Vereines Deutscher Ingenieure" for the matter from which this description and the illustrations have been prepared.

BRIDGE WORK ON THE KANSAS CITY, PITTSBURG & GULF R. R.

(With two-page plate.)

In the description of the Kansas City, Pittsburg & Gulf R. R., published in our issue of Feb. 17, we promised to give some further information respecting the bridge work on the line. In the present article we describe more particularly three specially interesting features of this work: (1) The Waddell patented form of 100-ft. through truss; (2) the concrete piers of the Arkansas River bridge; (3) the construction of these piers, with the engineering difficulties encountered and overcome.

From the tabular statement given in the former article it will be seen that the bridge work on the main line, 788 miles, amounts to 92,288 ft., or about 17.5 miles, as follows: Timber trestles, 83,462 ft.; truss bridges (including 1,471 ft. of the Arkansas River bridge), 5,927 ft.; plate girders (50 and 60-ft. span), 1,695 ft.; I-beams, 1,204 ft. Mr. Robert Gillham, M. Am. Soc. C. E., M. Inst. C. E., is Chief Engineer and General Manager, and Mr. J. A. L. Waddell, M. Am. Soc. C. E., was Consulting Bridge Engineer.

Steel Superstructures.

Plate Girders.—The plate girder bridges are of two standard lengths, 50 and 60 ft. span, both of which are deck bridges. The 50-ft. spans are 5 ft. deep, with web plates ⅜ × 59¾ ins., and top and bottom chords, each composed of two 66-lb. angles, 6 × 6 ins., and a cover plate ½ × 16 ins. The stiffeners are 24-lb. angle irons, 3½ × 3½ ins., those at the ends having fillers 9-16 × 4 ins., while all the others are crimped. At the middle of the span are inside and outside splice plates ⅜ × 12 ins. The rivets are of soft steel, ⅞-in. diameter. The girders are set 6 ft. apart, c. to c., and connected by four bracing frames, riveted to ⅜-in. connection plates. The two end frames have four 24-lb. angles (top, bottom and two diagonals), 3½ × 3½ ins., while in the two intermediate frames the bottom angle is omitted. A ⅜-in. plate is riveted at the intersection of the diagonals. There is also a single system (non-intersecting) of top lateral bracing of angles of the same size, also riveted to connection plates. At the sliding end, a 1¼-in. bearing plate, 20 × 20 ins., is secured to the bottom chord by countersunk rivets, and rests on a ½-in. bed plate, 21 × 21 ins. The anchor bolts are 1¼ ins. diameter, and the bolt holes in the bottom chord are slotted ½-in. on each side of the bolts to allow for expansion. The live load is Waddell's "Class X," and the dead load is 900 lbs. per lin. ft.

The 60-ft. girders are 6 ft. deep, with web plates 7-16 × 71¾ ins.; chord angles, 6 × 6 ins., 72 lbs.; cover plates, ⅝ × 16 ins., 42 ft. long; top lateral bracing, 30-lb. angles, 4 × 4 ins. The stiffeners, splice plates, and bracing frames are similar to these in the 50-ft. spans, but there are three intermediate frames. The girders are set 7 ft. apart, and the dead load specified is 1,000 lbs. per lin. ft.

Waddell's "A-Type" Truss.—This is a decidedly interesting style of truss, the designs for which have been patented by Mr. Waddell. It is a four-panel truss bridge, having eye-bars in the bottom chords and center verticals, and rigid members for all the other portions of the trusses and for the lateral system. In his "De Pontibus," Mr. Waddell states that for a number of years he was dissatisfied with all railway bridges for spans between the superior limit of the plate girder and a length of about 150 ft. He considered that ordinary through Pratt trusses were too light and vibratory, and that riveted bridges as then built were clumsy, unscientific and uneconomical. In 1893, when he was retained by the Kansas City, Pittsburg & Gulf R. R. to design some bridges, he persuaded the General Manager to have a "type A" truss of 100 ft. span built as an experiment, and this style of structure was soon afterwards adopted as the standard 100-ft. span of that railway. These bridges are also used on the St. Louis Southwestern Ry., and have been adopted by the Nippon Ry., of Japan, for spans of 65 ft. to 116 ft. The advantages claimed for this type of bridge are great rigidity in all directions, ease and cheapness of erection, and economy of metal when structures of this type are compared with structures of other types having equal strength and rigidity.

The 100-ft. spans of this type on the K. C., P. & G. Ry., are triangular trusses, 100 ft., c. to c., of end pins, and 40 ft. high at the center, as shown in Fig. 1. Each inclined top chord is built up of a cover plate ⅜ × 18 ins. (6.75 sq. ins. section), and two 15-in., 96-lb. channels (19.2 sq. ins.), 11 ins. back to back, laced below with bars ⅜ × 2½ ins. The bottom chord is composed of pairs of 1 × 5-in. eye-bars, 25 ft. long (10 sq. ins.), while the center vertical is composed of two eye-bars 1¼ × 5 ins. (12.5 sq. ins.). Each of the other two verticals is composed of two 10-in., 57-lb. channels (11.4 sq. ins.), set back to back 10 ins. apart, and connected by lacing bars ⅜ × 2 ins. The two diagonals are each composed of 10-in., 60-lb. channels (12 sq. ins.), 7⅝ ins., back to back, with similar lacing. The end pins are 5 ins. diameter, and the other pins all 4 ins. diameter. Fig. 1 shows the general elevation and plan of the bridge, while Fig. 2 shows the composition of the trusses.

The trusses are 17 ft. apart, c. to c., connected by top transverse struts at the three panel points, with diagonal bracing between these. The clear headway is 21 ft. 2 ins. Plate-girder floor beams, 4 ft. deep, are fitted between the bottoms of the three verticals, projecting but little below the bottom chord. These carry plate girder stringers 38 ins. deep, spaced 8 ft. c. to c. Horizontal and vertical bracing is fitted between the stringers, and the ends of the floor beams are also connected by diagonal bracing. The middle floor beam is riveted to hangers of two 10-in., 57-lb. channels, 10 ins. apart, connected by bracing, attached to the eye-bars by 4-in. pins.

The floor system consists of yellow pine ties, 8 ins. wide and 10 ins. deep, spaced 5 ins. apart in the clear and boxed out ½-in. over the chords of the stringers. Most of the ties are 10 ft. long, but every fourth tie is 14 ft. long, the ends supporting two foot planks 2 × 10 ins. Inside 60-lb. guard rails are used, 5 ins. from the track rails. There are also heavy outside guard timbers, 8 × 10 ins., laid flat and boxed out 2 ins. over the ties, which is an unusual depth of boxing, and is intended to prevent bunching of the ties by derailed wheels. These timbers, which are 8 ft. c. to c., are bolted to every tie, the ¾-in. bolts being in two staggered rows, and having ogee washers on top of the guard timbers.

The dead load is assumed as 1,400 lbs. per lin. ft. The live load is Waddell's standard loading, Class X, the equivalent uniform load per lin. ft. being as follows: 4,000 lbs. for trusses, 4,600 lbs. for floor beams, and 5,750 lbs. for stringers. The camber is such that under a live load of 4,000 lbs. per lin. ft. there will still be a camber of at least ¼-in.

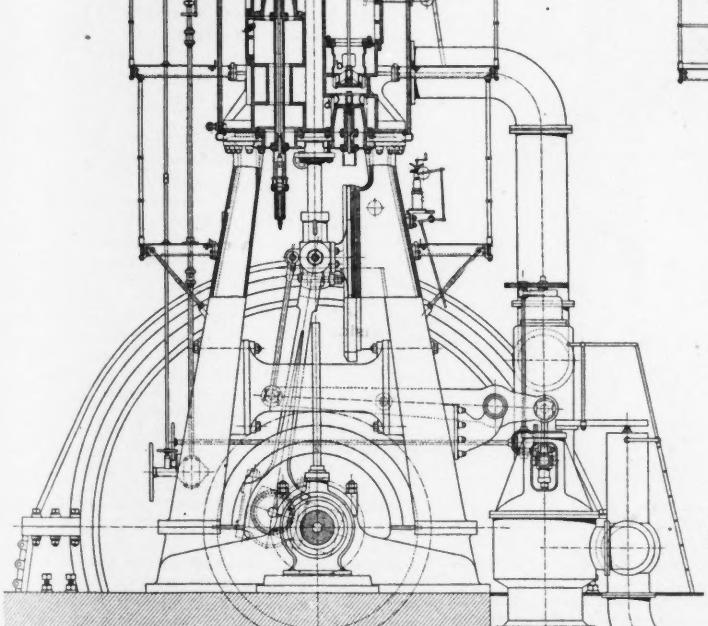


FIG. 2.—SECTIONAL END ELEVATION SHOWING DETAILS OF HIGH PRESSURE VALVES, CROSSHEAD AND AIR PUMPS.

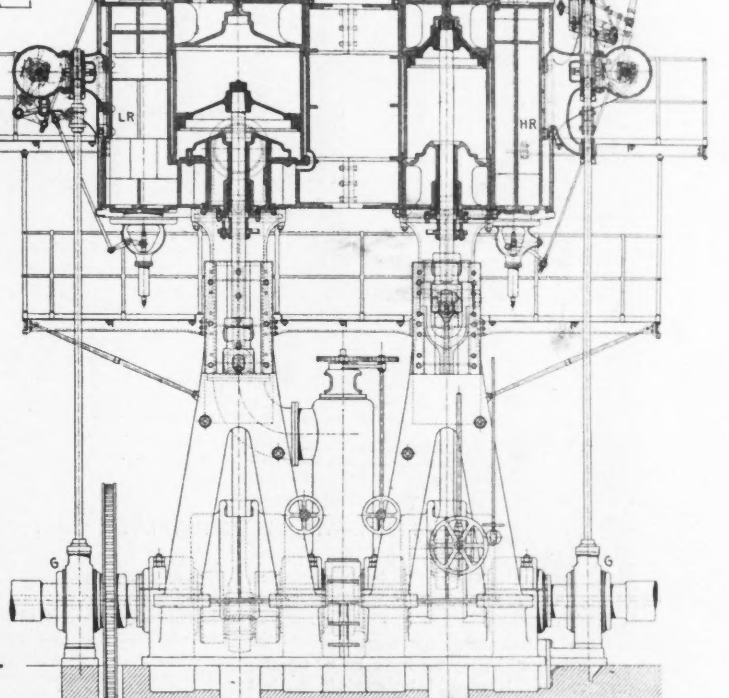


FIG. 3.—SECTIONAL FRONT ELEVATION SHOWING DETAILS OF CYLINDERS, RECEIVERS AND VALVE MECHANISM.

Through Truss Bridges.—The through bridges of 127 ft. and 150 ft. span are of interest in that the trusses have riveted connections instead of pin connections, as shown in Fig. 3. The inclined end posts and top chord are composed of two 15-in. channels, set back to back; the end posts have a cover plate on top with lacing on the bottom; while the chord has top and bottom lacing. The bottom chord and end verticals are of H section, built up of four angles with transverse stay or batten plates at intervals. All the other truss members are composed of two channels set face to face, and connected by lacing. Connection plates of ample area are used at all panel points, as shown, and knee braces connect the top lateral struts with the posts. The floor beams are plate girders 4 ft. deep, resting on angle brackets on the posts and riveted to the posts. The rail base is 4 ft. 4½ ins. above the center line of the bottom chords. The trusses are 28 ft. deep, c. to c. of chords, and have panels 25 ft. long. They are set 17 ft. apart c. to c., and have a clear headway of 21 ft. 6 ins.

The 200-ft. spans have pin-connected trusses. Special features of these trusses, as of the other trusses, are the substantial character of the design, the heavy floor beams and overhead bracing, and the substantial design of the connections.

All the material is of medium steel, put together with ⅞-in. soft steel rivets, except that ¾-in. rivets are used in some of the angles and lacing bars. The truss members were assembled in the shops and had the rivet holes reamed to a perfect fit. The rivet holes were punched ⅛-in. smaller than the rivets and then reamed to a diameter 1-16-in. greater than that of the rivet. The ends of all stringers and floor beams, and of all abutting members, are planed, so as to have a perfect contact. The metal work was given one coat of paint at the shops and two after erection in the field. The general particulars of 150-ft. and 200-ft. spans are as follows, the loading being all per lin. ft. of span and the live loads being Waddell's Class X:

Length, c. to c. of end pins	150 ft. 0 ins.	200 ft. 0 ins.
Panel length	25 " 0 "	28 " 6⅞"
Truss depth at center	28 " 0 "	37 " 7 3-16
Distance between trusses, c. to c.	17 " 0 "	17 " 0 ins.
Clearance above rail base	21 " 6 "	21 " 6 "
Dead load for trusses	1,700 lbs.	1,920 lbs.
Dead " floor system	700 "	700 "
Live " trusses	4,140 "	4,020 "
" " floor beams	4,900 "	4,720 "
" " stringers	5,925 "	5,720 "
Wind load for top lateral system	150 "	200 "
Wind load, bottom lateral system	450 "	500 "

Arkansas River Bridge.—This bridge, shown in Fig. 4, is the longest on the line, aggregating 1,471 ft. It has a through truss channel span of 250 ft., flanked on the north by five deck trusses (4 of 127 ft. and 1 of 147 ft., c. to c. of end pins) and a plate girder of 50 ft. span. On the south there are three deck trusses of 147 ft. and a plate girder of 50 ft. span. The high water elevation is 31 ft. above low water elevation, and the base of the rail is 60 ft. 6 ins. above low water.

Concrete Piers; Arkansas River Bridge.

The ten piers and two abutments of the bridge over the Arkansas River are all built of monolithic concrete, and in the construction of the piers numerous difficulties were encountered. The following table gives the dimensions and cubic contents of the piers:

Concrete Piers of the Arkansas River Bridge; K. C., P. & G. R. R.

No.	Height, ft.	Height, ins.	Length and breadth — At base — ft. ins.	Length and breadth — At base — ft. ins.	Length and breadth — At top — ft. ins.	Length and breadth — At top — ft. ins.	Low water, cu. yds. — Above.	Low water, cu. yds. — Below.
Abut. A..	33	3	53 8	× 10 5½	12 0	× 5 0	418.85	.....
Pier 1..	46	9¼	24 0	× 8 6	21 6	× 6 0	243.20	47.47
" 2..	46	4½	24 0	× 8 6	21 6	× 6 0	243.20	44.68
" 3..	49	5¾	24 0	× 8 6	21 6	× 6 0	243.20	66.32
" 4..	65	3¼	30 4	× 14 0	23 6	× 6 6	479.59	138.17
" 5..	64	4¼	29 0	× 12 0	23 6	× 6 6	479.59	107.08
" 6..	36	7½	24 8	× 8 6	21 6	× 6 6	219.81	.....
" 7..	20	4	24 0	× 8 6	21 6	× 6 6	125.14	.....
" 8..	21	0	24 8	× 9 2	21 6	× 6 6	111.29	.....
" 9..	20	6	24 8	× 9 2	21 6	× 6 6	107.21	.....
" 10..	20	10½	24 8	× 9 2	21 6	× 6 6	110.52	.....
Abut. B..	40	4¾	19 4	× 10 0	12 0	× 3 0	139.94	.....
Total							2,921.54	403.72

Mr. A. M. Nelson was engineer in charge of construction at this bridge, and in a report made to Mr. Robert Gilham, the Chief Engineer and General Manager, he gives an interesting account of the methods of construction, and the methods of overcoming the difficulties. An abstract of this report we give below, believing it to be of special interest and value. It should be explained that abutment A and pier No. 1 are at the south end of the bridge, the enumeration running from south to north. Fig. 5 shows the details of these piers and abutments:

Surveys.—The positions of the piers were determined by triangulation, for which four independent base lines were used. On the south side of the Arkansas River a perpendicular was erected, extending 1,500 ft. east and 1,272 ft. west of the center line. From this double base line the distance across the river was calculated, and checked by calculation from another line thrown off at random from the bridge tangent, at a point that afterwards proved to be 10 ft. 1 in. south of the center of pier No. 8. This measurement was subsequently checked again by another line thrown off at right angles to the center of pier No. 10, and extending 2,000 ft. to the east.

Several points were established on each of these base lines, and angles read to and from each of these points, all of which were read and re-read 20 times, giving direct interpolation of the angle to within 3 secs. From this close reading of the angles, and the accuracy with which the base lines were measured, the various calculations checked within 0.026 ft.

The base lines were measured in the following manner: We first used an ordinary 100-ft. steel tape line, supported at intervals of 25 ft. upon the tops of stakes which were driven to a perfect level. This measurement was afterwards checked with a wooden rod 20 ft. long, and measured to knife blade scratches.

At corresponding distances from the triangulation points and the tangent—that is, distances corresponding to the distances from the initial point to the centers of the piers—piers were staked out on these base lines, so that before the construction of the piers progressed far enough to obstruct the view, cofferdams and cribs could be set by direct sight. In all cases when it was impossible to see across the river to any point, the angles could be turned so that a right angle intersection could be made on the direct points in the stream, locating the exact center or points corresponding to the points used on the shore. These angles were all of 45°, thus making it unnecessary at any time to have to refer to field notes, or for any purpose carry maps or plans or notes of the survey into the field. At any time, and under any circumstances or conditions, points could be given by direct intersection from some of the number of base lines established.

The masonry was begun on both sides of the river, and when all the piers were completed and the falsework placed in position, so that direct measurements were obtainable, the measurements checked out as close as it was possible to measure, illustrating the care and accuracy with which the former work had been done. All points on shore were additionally referenced with hubs and guard stakes, so that at low stages of the water the points for piers Nos. 3, 4 and 5 could be easily established from these intermediate references.

In addition to this, we placed at right angles from each of the three above-mentioned piers, range stakes, to enable the contractors to float their cribs and cofferdams into position without having to wait for the engineer to give them lines or points. These ranges were so arranged that the intersection of the lines would bring them in direct position and could be used by the contractor's foreman by day or by night.

The triangulation points, ranges and all other points were preserved without very much difficulty through the construction of the work.

Concrete.—On June 17, 1895, the Arkansas Construction Co. entered into a contract with McGee, Kahmann & Co., of Kansas City, Mo., for the construction of the piers. Work was to be commenced immediately and completed by Oct. 1, 1895. The 10 piers and 2 abutments were to be built of Portland cement concrete according to plans and specifications furnished by the Arkansas Construction Co. The concrete was composed of 1 part Portland cement, 2½ parts of coarse sharp sand, and 5 parts of clean broken stone. The sand and stone were found on the site of the bridge. The cement was to be of the best quality of hydraulic Portland cement. Although the Arkansas Construction Co. showed a preference for North's Portland cement, no first-class brands were barred. Subsequently two brands of cement were used, North's Portland (Condor brand) and Kelley Island Cement Co.'s Lagerdorfer (Tower brand), both filling the requirements of our specified tests, and giving good results in the work.

The contractors moved onto this work about July 1, and began the preliminaries, although no actual construction was done until Aug. 22, when the first concrete was mixed and placed in the mold or form of pier No. 7. On March 6, 1896, the last batch was hoisted into the form of pier No. 4, and on July 22, 1896, the plastering of all of the piers and abutments was completed.

Molds or Forms.—The forms were all made of 2-in. yellow pine board surfaced on one side, and sized to 1¾ ins. The semicircular ends each tapered at the rate of 1 in. per ft., and were made by first making a double segmental set of ribs, placed 5 ft. apart vertically, and spiking timbers, 2 × 6 ins. (which had been tapered to fit these circles), to the segmental ribs. At each end of these ribs were angle irons bolted to the vertical stanchions, which were 4 × 6 ins., and answered as corner posts of the form.

Between these corner posts there were two additional vertical posts supporting the sides or flat surfaces of the form. They were sheeted up to the inside with yellow pine boards, 2 × 12 ins. These posts were about 4 ft. c. to c. and were held firmly in place by having at intervals of 5 ft. perpendicularly a ¾-in. iron rod, passing through a 1½-in. gas pipe which was fitted inside of the form, the gas pipe acting as a strut, while the rod on the inside was in tension. The construction of the molds is shown in Fig. 6.

These forms were placed in position with the transit instruments, and were watched closely and frequently checked. There being no means to brace the form in the middle of the river except from the interior, it made it a little difficult to keep them in position. This was done, however, by using X-bracing on the interior of the semi-circular ends, and removing it when the concrete would reach the bottom of the braces. After the piers were completed, the gas pipes, of course, remained within the concrete; upon taking the forms apart the rods were easily removed from the interior of the gas pipe.

On the forms of piers Nos. 1, 2, 3, 4, 5 and 6, at the point where the straight barrel of the pier meets the battered line of the upper portion of the structure, there is built with the pier a projecting mold to hide this union as well as to trim and beautify the structure. The form for this molding, Fig. 7, was made by sawing the form blocks with a scroll saw, and fitting cottonwood strips, ½ × 1½ ins., which had been previously soaked in hot water to make them bend readily to the form blocks. This

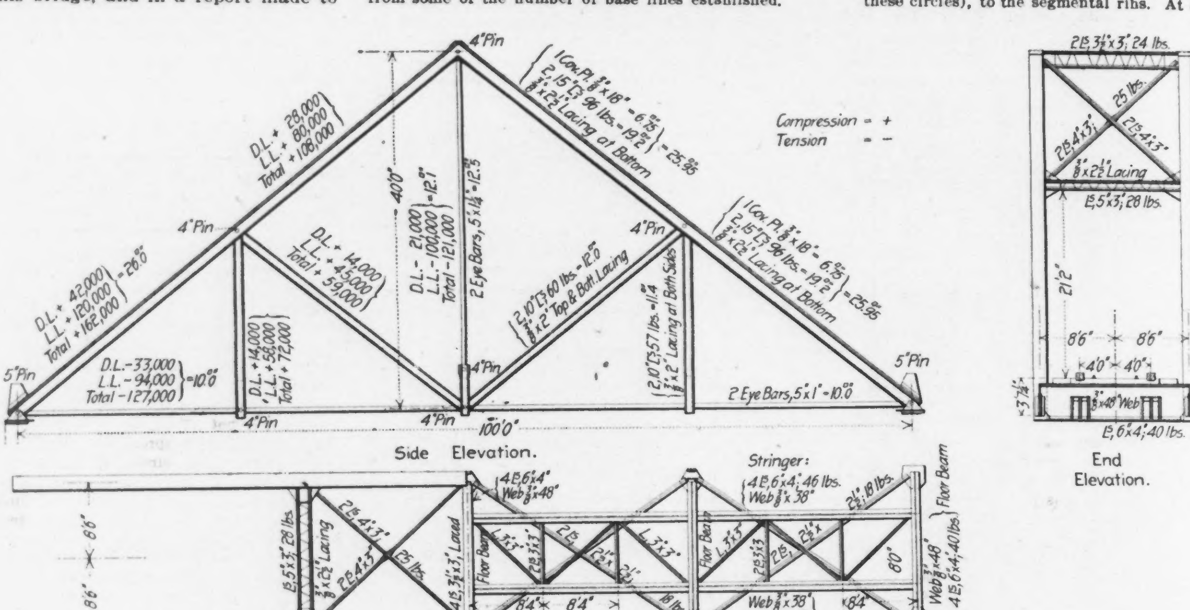


FIG. 1.—WADDELL'S PATENTED "A" TYPE OF THROUGH TRUSS FOR SHORT SPANS.

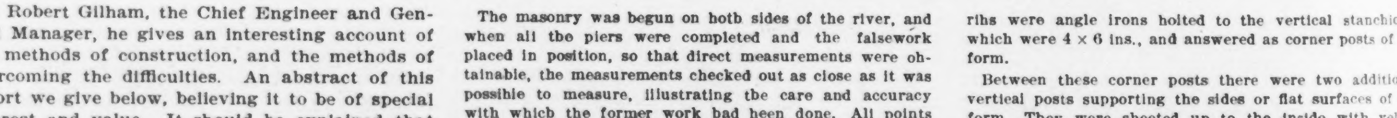


FIG. 4.—ELEVATION OF ARKANSAS RIVER BRIDGE; K. C., P. & G. R. R.

molding was also used on both abutments, and adds very much to the appearance of the structure.

Abutment A.—This is built on sand 7 ft. below the surface of the ground, and is the only one of the structures that is not on solid rock throughout. This abutment, having a broad base and circular wings, was not intended or designed to go down to solid rock, and a year after it was first commenced it had not shown any signs of settling, although it supports an embankment about 30 ft. high, which was placed against this abutment as soon as the form was removed, or within ten days from the time the last concrete was put in the forms.

On Oct. 7, 1895, the base course was put in, which is one foot larger than the neat size of the abutment in all directions. On Oct. 8 and 9, the form for the abutment proper was put in place, and the concreting continued until Oct. 12, when the work was stopped until the foundation of pier No. 1 could be secured, which was then in quicksand. On Nov. 13 work was resumed on the abutment, and it was completed (except the plastering) Nov. 20. The form was removed Nov. 27 to allow the grading contractors to finish up the embankment; and the plastering on the back of the abutment and the portions of the face to be covered with dirt was finished as soon as the form was removed. The balance of the abutment was plastered and finished Dec. 13. All of the coping on the top of the wings is made of concrete and the bridge seat proper is Carthage limestone.

Pier No. 1.—This was the most troublesome of all the piers, notwithstanding the fact that it was built on the bank (practically on dry land). It began to give trouble from the commencement of the work, showing signs of caving and sliding in the excavation. Later on the excavation was carried down to the level of the sand bar, or an elevation of 408, and then a frame 3 ft. larger than the pier was placed in position, and round piles 2 ft. apart were driven to the bed rock on the outside of this crib or frame. The whole excavation was shored to keep from caving, but when the excavation reached the water level, or an elevation of 402, part of the shoring gave way and a number of men narrowly escaped being buried.

The scheme of the contractors was to sheet up the piles on the inside as the excavation progressed. This worked very well until the water was encountered, and then the sand came in so rapidly that it was impossible to use the longitudinal sheeting any further. The contractors then concluded to drive small Wakefield piles and put the frame on the inside. The excavation was carried on in this way, and the piles driven down as the excavation progressed, until quicksand was struck, when another line of small Wakefield piles was driven, and the quicksand excavated by means of shovels until the sand began to cave in around the outside of the crib. This left large crevices, which were filled with hay, sheaf oats, sorghum stalks, willow boughs, stable manure and other material, in fact anything that would stop leaks. In some cases burlaps and everything available that would stop a crack between the boards was used. The manure and hay eventually worked through the openings between the piles, and very seriously interfered with the pumps in the foundation, many times causing them to be stopped. Eventually the contractors worked the quicksand down within 18 ins. of the bed rock, where they worked day and night for 18 days without lowering the quicksand at all.

A plan was then suggested that finally worked well, and enabled the foundation to be speedily secured. It was as follows: Small cofferdams or minnature forms, 2 × 4 × 2 ft., were made, and placed one at a time along the outside of the neat work, allowing a slight projection of the small form outside of the neat lines of the main forms. These small forms were made with water-tight sides and open top and bottom, so that they could be easily removed after the concrete inside had had sufficient time to set. These forms were placed in the quicksand, 1 ft. apart, allowing the water and quicksand to pass between them in its flow to the pumps, and then the forms were worked down to the bed rock by baling out all of the sand and water from inside them. As fast as one was filled with concrete another would be sunk, baled out and filled, and so on until the circle was completed. When the small forms were all in and filled there was space left between the sides of these small forms and the crib proper for the passage of the sand and water. Next the 1 ft. passageways were to be closed, but it was then apparent that some means must be employed to allow the free flow of water and sand to the pulsometer pumps, which stood in the center of the pier, about 5 ft. from the east end; so for the time being two channels were left, one running north and south and the other east and west, intersecting at the pump. After the form for the pier was in place, it became necessary to put a form around the pump, in order to build up the concrete and leave the pump in a sump or cistern until the pier could be built above the danger line, or the present water mark.

The next step was to confine the water and quicksand to the sump. This was accomplished by putting in a 6-in. wrought iron pipe to connect the outside passageways directly with the pump, and exclude the sand and water from the inside of the form, except that which passed through the pipe. This was done in both channels, and the work of sealing the foundation then commenced.

In the 12-in. spaces left between the small forms, one Wakefield pile was inserted on either side, making a tight joint with the other Wakefield piles that had been driven around the small forms previously, and left there after the completion of this connection. The sand on the bed rock on the interior of these small spaces was then taken out, and the concreting of these openings was then carried out.

All the concrete in this foundation, up to elevation 407, was put in place by means of a chute. The concrete was wheeled in with wheelbarrows into the form, and shot into position to prevent separation of the mixture. The first concrete for this pier was mixed and put in position on Nov. 9, 1895, and the pier was completed on Nov. 23. The form was removed Dec. 2, and the pier was plastered soon after the removel of the form. The coping was set about Dec. 10.

The bank is filled in around the pier as it originally was, and reveted with an 18-in. dry masonry wall, laid with the slope of the bank, which is 1 on 1½. The wall contains 143.7 cu. yds. of masonry, and was let at $2.75 per cu. yd.

Pier No. 2.—This was the easiest and simplest of the piers in the bed of the river. The foundation was secured by driving Wakefield piling around a crib 3 ft. larger than the foundation. This piling is shown in Fig. 8. Some of these piles reached the bed rock, but many of them did not. As the excavation progressed, shores were inserted to protect the sides of the cofferdam. When the excavation reached the lower ends of some of the piles which the contractors failed to get down, then another frame was inserted and another row of small Wakefield piles was driven, and then before bed rock was reached still another row of Wakefield piles was driven.

This proved successful, and concreting was commenced on Oct. 26, 1895, and completed Nov. 3; this pier was

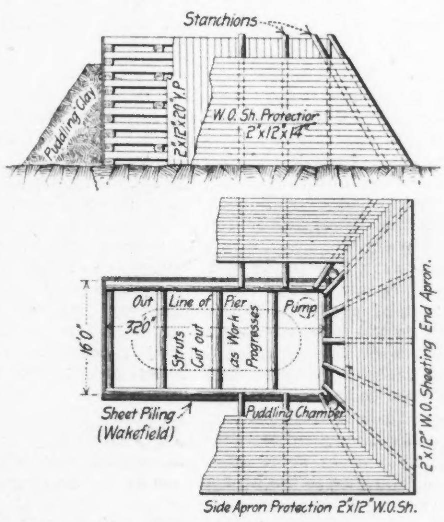


Fig. 9.—Crib and Apron for Pier Foundation.

partially plastered in December, but owing to the cool weather the plastering was discontinued until May, 1896, when this pier was finished up. It contains 287.88 cu. yds. of concrete; 44.68 cu. yds. of which are below, and 243.2 cu. yds. above an elevation of 400.

During the floods in the Arkansas River, when the river reached an elevation of 429.87, this pier had at least 4½ or 5 acres of drift accumulated against it, with a current in the river at the time of eight miles an hour, with no weight on the top of the pier to counterbalance the pressure against it. After the water had receded the pier showed no signs of having been affected by the pressure of the stream and the drift.

Pier No. 3.—This pier was constructed by building a crib of round timbers, flat on the outside and placed in position in about one or two feet of water, and driving Wakefield piles on the outside of the crib, and then placing, by means of a diver, about 1,000 burlaps filled with sand about 10 ft. out from the bottom of the crib. These burlaps were for protecting the crib, and they were hauled in by means of barges. The whole crib was puddled with stiff puddling clay. Numerous small rises in the river from local rains gave considerable trouble, and it was with considerable difficulty that this crib was made to resist the pressure of the water from the outside, and quantities of hay and cane stalks were used to stop the leaks. Finally an auxiliary cofferdam was driven to protect the puddling from the current, after which bed rock was reached. The foundation for the pier was dry, and all dirt and slime was wiped off of the rock bottom with burlaps. There were a few loose flat stones, which were removed with little difficulty.

The character of all the river foundations was a grayish blue sandstone, very hard and rough, and with only an occasional seam. The general surface of the bed rock is quite level.

The pier was commenced by putting the first concrete in position Nov. 18, and was finished Dec. 6, 1895.

The form was removed Dec. 13, and the pier withstood the floods and drift of Dec. 19 and 20, 1895, during which trees and logs struck the fresh concrete and left no traces of the effects after the flood had receded.

Pier No. 4.—This was the last pier constructed, and many difficulties were encountered during its construction, as the bed rock was scoured by the current of the river and was perfectly bare. A crib was made of round logs, flat on one side with the flat side turned out. This crib was subdivided into four parts or chambers, and loose struts were used to separate the chambers as well as to resist the pressure of the forces from the outside. Wakefield piles were then driven around this crib and sheet piling driven outside of that about 10 ft. away, forming a chamber between the sheet piles and the Wakefield piles, which was filled with puddling clay. The sheet piles were also puddled on the outside, and protected by sand bags. Numerous breaks occurred in this cofferdam, and considerable annoyance was caused by unexpected rises in the river.

Finally, when all was in readiness to begin the concreting, the rains set in and heavy floods occurred all over the country, especially in the valley of the Arkansas and streams tributary to it. They were swollen and overflowed their banks, and the Arkansas River rose rapidly and washed away the cribs, sheet piles, pumps and everything within its banks. All the sand that had been placed on the Cherokee shore, and the broken stone for the whole of pier No. 4, and about one-third of pier No. 5 (about 1,200 cu. yds.), was washed away.

The contractors' pumps, hoisting engines and other machinery, tools, etc., were all left in the bottom of the river. This very naturally affected the progress of the work. This flood occurred Dec. 19, and the water did not recede until after Jan. 21, 1896, when another crib was built similar to the one above described, and the same style of Wakefield piles was used, except that they were longer and the crib was built 4 ft. higher, so that small rises in the river could not affect this foundation work. This crib was protected with aprons built of 2-in. oak planks, set at an angle in all directions, giving chambers between the crib and the aprons an enlarged base. This aperture between the crib and the apron was filled with puddling clay, and a great deal of hay, burlaps and other material was used in connection with the clay to stop leaks in the cofferdam. This construction is shown in Fig. 9. The foundation, on bed rock, was rather smoother than the others, having no loose material on it whatever. The concreting was commenced Feb. 8, and finished March 6, 1896. This pier has an enlarged base, as shown in Fig. 5. It contains 617.76 cu. yds. of concrete, 138.17 of which are below and 479.59 above an elevation of 400. The stone for this pier was all broken by hand, the crusher having been shipped away before the flood.

Pier No. 5.—This is the channel pier on the Cherokee side, and was constructed by using a separate cofferdam made of yellow pine timbers, 2 × 12 ins., 16 ft. long, except the bottom and top fillers, which were 10 × 12 ins., 16 ft. long. The cofferdam was rectangular in shape, 32 × 16 ft., and 18 ft. deep; it was sheeted upon the inside and outside with 2-in. pine boards running longitudinally, and calked with two threads of oakum. The vertical studs, 2 × 12 ins., 18 ft. long, were placed 2 ft. apart. This left chambers or recesses, 12 × 24 ins. between the studs, to be subsequently filled with sand to sink the crib; the recesses being left open at the top for that purpose.

The two parts of the cofferdam joined at each end, the joints being provided with cushions made by tacking canvas facing to the edges loosely, and stuffing them with oakum, making a cushion about a foot thick. The bottom of this cofferdam was built to conform to the contour of the bed rock, which was bare and very rough. At a point 3 ft. above the bottom an additional canvas apron about 12 ft. wide was secured to the sides of the cofferdam by nailing strips 4 ins. wide to prevent the canvas from tearing. This canvas was stretched out horizontally and weighted with sand bags, and also with puddling clay.

The contractors abandoned the idea of loading the pockets of the cofferdam with sand, and subsequently loaded the top with burlaps filled with sand, and the canvas apron was loaded with sand bags for 12 or 15 ft. Puddling clay was used, and the whole exterior of the cofferdam was thoroughly puddled. This cofferdam did not prove a success. There was no struts placed inside before the pumps were started, consequently as soon as the water was pumped out the cofferdam sprung in at the center, all along the sides, causing great leaks. The pumps had to be shut down, and divers sent inside to place shores to resist the pressure of the water. Sand bags were also used at the bottom to stop leaks temporarily, as well as keep the lower edges of the cofferdam from further bulging in or slipping. After much time had been lost, the cofferdam was thoroughly braced inside and the foundation for this pier was secured. The bed rock showed a crevice about 2 ft. wide running diagonally across the east end; this was excavated to a depth of 6 ft., when it disappeared. The character of this foundation is bluish gray sandstone, and this crevice seemed to be simply a crease in the solid rock. Concreting was commenced on Dec. 6, 1895, and completed Jan. 31, 1896. On Dec. 18 the pier was built to an elevation of 134.5. The high water then set in, and nothing more was done until Jan. 29. During this flood a portion of the form was torn away by drift wood, but the pier was unhurt. The plastering on this pier was completed July 22, 1896.

Pier No. 6.—This has a foundation above low water mark, but at the time of construction the water was about 2 ft. over the foundation. A half crib was used, a double tarpaulin being fastened to the top of this half crib and extended out in the river about 12 ft. This was puddled with clay and the water kept out. The solid rock sloped at a considerable angle in this foundation, and it had to be blasted to a level. The base of this pier was put in to receive the form on Oct. 28, 1896, when the concreting was commenced. The pier was finished Nov. 5, 1895, and the pier was plastered in June, 1896. In July the river was low enough to permit working around the base, when the Arkansas Conctruction Co. put in 17½ cu. yds. of Portland cement concrete reinforcement around the base, completing the pier, which contains 219.81 cu. yds. of concrete, all above an elevation of 400. This does not include the reinforcement done by the Arkansas Construction Co.

Pier No. 7.—This was the first pier built. It is a shore pier on a solid gray sandstone foundation, only about 18 ins. of excavation through loose rock being necessary to reach the foundation. It was commenced Aug. 23, 1895, and completed Aug. 26, 1895. The pier has a reinforced base, this reinforcement being done by the contractors.

Pier No. 8.—This was the second pier constructed, and the foundation for it is similar to that of No. 7, except that it is deeper to the solid rock. This pier was commenced Aug. 21, and completed Aug. 31, 1895, and contains 111.29 cu. yds. of concrete in the pier proper. It was plastered in December, 1895. The weather turned cold just after this pier was plastered, and a quantity of the plaster was loosened by the action of the frost. In July, 1896, just before the plastering was all completed on the other piers, this pier had all of the loose plaster removed, and was replastered and washed down with a strong solution of cement mortar or liquid cement. The high water of December, 1895, tore up a quantity of loose rock around this pier and exposed the foundation of the east end. In July, 1896, the Arkansas Construction Co. reinforced the base of this pier by putting 24 cu. yds. of concrete around it.

Pier No. 9.—This has the same kind of a foundation as No. 8, except that it is still deeper to the solid rock, and the loose rock through which the excavation is made has a tendency to be more solid, the last 18 ins. of this excavation being through hard, blue slate. This pier contains 107.21 cu. yds. of concrete. The concreting was commenced Aug. 28, and was finished Sept. 26, 1895.

Pier No. 10.—This is built on a hard blue slate foundation, perfectly smooth and solid. The concreting was commenced Sept. 9, and was completed Sept. 24, 1895. This pier has an embankment about it, which is protected by a rip-rap wall above high water mark.

Abutment B.—This abutment, at the north end, is on a hard blue slate foundation, which is level, smooth and solid. The concreting was commenced Aug. 28, and was completed Oct. 28, 1895. This abutment contains 139.94 cu. yds. of concrete and was plastered two coats, floated on with a cork float. The steel float and the third coat were not used except on that portion exposed above the embankment, which is 6 ft. below the top of the abutment. This piece of masonry is covered by an embankment, so that only the top is exposed to view.

STEEL TIES ON THE NETHERLANDS STATE RAILWAYS; 1881 TO 1898.

The use of metal ties was commenced on the Liege & Limburg line of the Netherlands State Railways in 1865, and since 1880 very careful observations have been made of the results obtained with the different types of metal ties which have been in service. This line is on Belgian territory and will soon be transferred to Belgian ownership, under an agreement between the governments of the Netherlands and Belgium, by which each country will purchase those lines within its territory which are now operated as a part of the railway system of the other country. Under these circumstances, the observations as to the metal ties on the Liege & Limburg line will be discontinued, and the Resident Engineer, Mr. Ch. Renson, has therefore prepared a final report on the general results of these observations for the 17 years ending with Jan. 1, 1898. This report will be published in full in the July number of the Bulletin of the International Railway Congress, and we present herewith an abstract containing some very interesting and instructive information. In our issue of April 7 we presented an abstract of a report on the experience with steel ties on the Gothard Railway, of Switzerland, and discussed the relation of the steel tie question to the track of American railways. It is to be noted that on both these foreign lines, where the ties have been used extensively and under proper supervision, very satisfactory results have been obtained both in point of efficiency and economy. The following is the abstract of Mr. Renson's report:

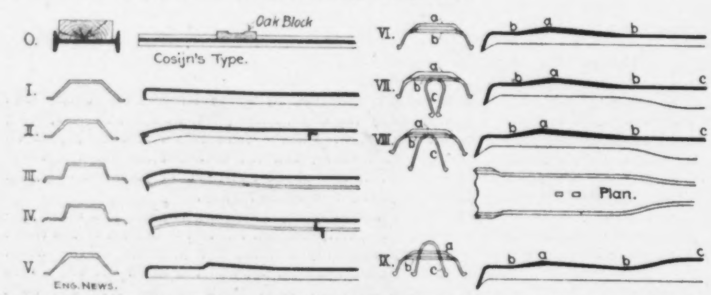


FIG. 1.—STEEL TIES ON THE NETHERLANDS STATE RAILWAYS; TYPES 0 TO IX.

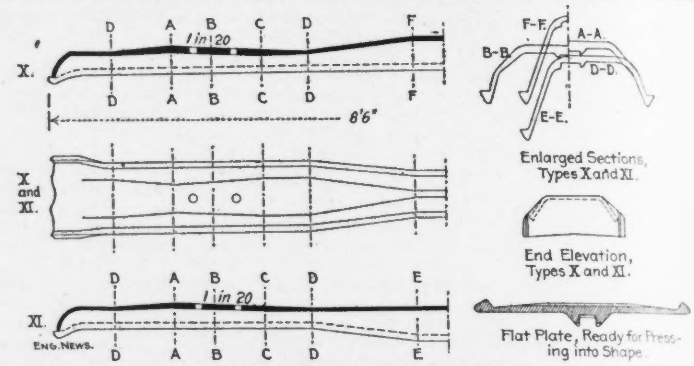


FIG. 2.—POST'S IMPROVED STEEL TIES; TYPES X. AND XI.; NETHERLANDS STATE RAILWAYS.

The first trial section of track with steel ties was laid on the Liege & Limburg line in 1865, but in 1880, Mr. J. W. Post, Engineer of Permanent Way of the company, which operates the state railways, was commissioned to study at home and abroad the results obtained from the use of metal ties, and also to inquire into the latest improvements in their manufacture. This investigation resulted in a series of extensive and practical tests, which have been continued in a systematic manner during the past 17 years. The trial sections are 27 in number, and have carried from 100,000 to 150,000 trains.* The present report shows the relative merits of the different systems tried. Up to 1888 ties of types I. to IX., Fig. 1, with fastenings of types A, B and C, Fig. 2, had been put in service. Since then, types X. and XI., with holes drilled instead of punched, and fastenings D, have been introduced, as shown in Figs. 2 and 4. Type O is now obsolete, but most of the ties of this type, which were laid in 1865, are still in use, but their wooden blocks have been replaced by cast-iron blocks. Types I. and II. are of iron, the others are of steel.

The 27 trial sections of track are on single track, laid with T-rails, the steepest grade being 1.6%, and the sharpest curve 1,148 ft. radius (5°). The traffic carried is from 14 to 25 trains per day, and 29 trains per day over one of the sections. In the accompanying table, the cost of maintenance per mile, and per 10,000 trains, is given for 21 sections, the others not being long enough for statistical purposes. The section No. 1, with oak ties, is a basis of comparison for the ten sections having 25 or 29 trains per day, but not for those with only 14 trains per day, as the relative cost of maintenance is higher, owing to the fact that these expenses are affected by the number of trains as well as by the time the track has been exposed to frost, rain, etc. The figures relating to sections with the lighter traffic are given in parentheses. The expenses are given in labor instead of money, which allows of their being compared with those of railways in countries having a different system of currency.

From my observation of the 27 trial sections during 17 years, and from the statistics of maintenance expenses of 21 of these sections, I present the following results:

The ties with punched holes show, after three or four years' service, little cracks beginning in the corners of the holes (rectangular with rounded corners) and caused by the punching operation, which usually bruises the metal, even in the case of mild steel. These cracks, generally invisible at first, are gradually enlarged by the train traffic. Ties with drilled holes do not crack in this way.

The ties of type I. last longer than oak ties. Calculated at the actual prices, the annual charge for renewal of these ties is only half that of oak ties. The maintenance expenses of tracks with tie I. are higher than those of oak ties; but if the difference is deducted from the difference in annual charge for renewal, there is a surplus of about $43 per mile of track in favor of this tie. It must be observed that type I. is the smallest, lightest and worst of the types tried and is now considered decidedly inferior. The work expended on the inferior systems of fastenings, A and C, had an unfavorable influence on the cost of maintenance of the ties which have these fastenings. For tie I. the maintenance expenses were higher than those for oak ties; for tie II. the difference was less, and so on; each following type being an improvement on the former. For the Post tie of type VI. those expenses are less than for oak ties, and the result is still more favorable for the improved Post types VII. to XI.

The rusting of steel ties is insignificant, even if the roadbed consists of cinders, as is the case on the greater part of this line. Of how little importance the rust is, is clearly shown on another line of the company. In 1865, 10,000 ties of the Cosijns type, Fig. 1 (iron I-beams laid flat and having two oak blocks), were laid between Deventer and Olst. About 200,000 trains have passed over these "venerable" ties, now 33 years old, but they are expected to last much longer, for the oak blocks, which only lasted three to eight years, are now being replaced by iron blocks.

A very important point, which till now has not attracted sufficient attention, is the way in which the tie affects the rail flanges, especially in regard to local wear of rail-flange, tie and fastenings. The rail being much better fastened on the metal ties, the hammering of the rail under traffic (often observed on wooden ties) is avoided. By measuring two series of rails, which had been 14 and 16 years in service, it has been found that on metal ties the local wear of rail, tie and fastenings is insignificant.

When I discovered the little cracks in the metal ties (with punched holes), Mr. Post designed the fastenings of type D (Fig. 4), permitting the use of bolts without a square collar, the head of the bolt being retained between two fillets rolled on the under surface of the tie (Figs. 2 and 4). With these

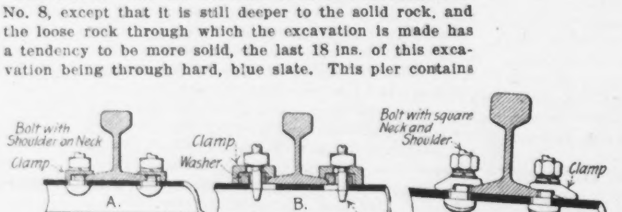


FIG. 3.—FASTENINGS FOR STEEL TIES; TYPES A, B AND C.

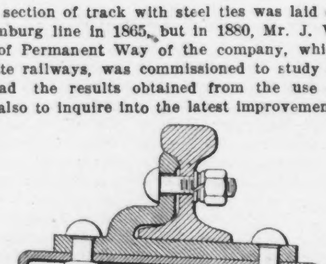


Fig. 5.—Renson's Rail Brace Chair for Steel Ties on Sharp Curves; Netherlands State Railways.

*Descriptions of the forms of ties experimented with and details of the results obtained with these ties may be found in the following publications:

"Maintenance Expenses of Track on Metal Ties," by J. W. Post, "Transactions of the American Society of Civil Engineers," June, 1888.

"Report on Metal Ties," made by Ch. Bricka to the French Minister of Public Works, 1886.

"Report on Metal Ties," made by J. M. Kowalski to the International Railway Congress, and published in the "Revue Generale des Chemins de Fer," February, 1886.

"Report on Metal Ties," made by E. E. Russell Tratman; Bulletins Nos. 3, 4 and 9 of the Forestry Division, U. S. Department of Agriculture, Washington, D. C., 1889, 1890 and 1894.

round bolts the holes in the tie must be round and can be drilled.

The great difficulty was to induce the steel works to put in the plant necessary for exact, quick and cheap drilling of the four holes per tie. There was some opposition on that point; just as there was at the time when the railway officers wanted to get the holes in steel rails drilled instead of punched. It was only in 1889 that a reasonable contract could be made.

The Post ties X. and XI. (Fig. 2), with fastenings D (Fig. 4), which have been in the track now since 1890, are superior to any of the former types.† They not only require very little maintenance work; but whereas types I. to IX. show cracks after three or four years' service, not one of the types X. or XI. (and they have been carefully examined) show a trace of cracks.

I prefer type X. with "dromedary" waist (Fig. 2), to type XI. with low waist, because with type X. the middle of the tie is not bedded in the lower and firmer part of the ballast, and so the support of the tie is sure to be under and near the rail seats.

A series of these ties, which were examined in 1897, show that the wear of the rail seats of the tie is insignificant and that the wear in the holes also is of no consequence.

For lines with very heavy traffic, however, it may be economical to still further lengthen the life of the tie by applying between the rail base and the tie a mild steel plate, say 10 or 12 mm. thick. In order to examine whether this might cause any reduction in the firmness of the fastening of rail to the tie, I armed some ties of types X. and XI. (fastenings D) with such tie-plates as shown in Fig. 4, and I found that there is no objection to their use, the gage of the track being as constant with these plates as without them.

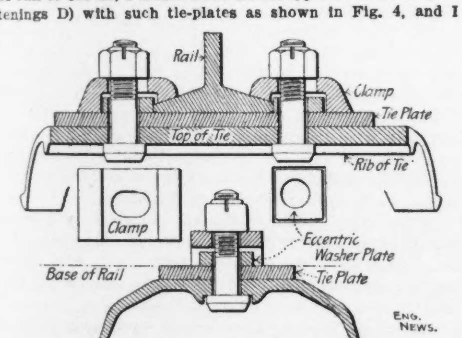


Fig. 4.—Post's Improved Fastening for Steel Ties; Type D. (Showing Tie-Plate.)

These plates can easily be renewed when worn and thus practically lengthen the life of the ties indefinitely, even on lines with heavy traffic.

For very sharp curves in main track, I have tried since January, 1890, a special tie with stamped iron chairs (Fig. 5), which is rather expensive, but gives satisfaction in every respect and is superior to any special system tried on the sharp curves of our lines. Rivets of 30 mm. applied by machine would be preferable to those of 25 mm., as used on the trial track.

As my impressions after these 17 years of experimenting, statistics and observation, I can give the following conclusions:

†The ties with variable section are either rolled in trough shape or they are manufacturd by rolling a flat bar (with variable thickness in some parts,Fig. 2),which is afterwards pressed into trough shape in a matrice. With the same section under the rail base, the economy of material in comparison to constant section is about 15% for the trough shape rolling; it is less with the flat rolling system. It is a long time since these rolling systems were patented; the patents are probably expired and public property now.

Grave defects have been discovered, especially the cracks in types I. to IX., and also imperfections in the fastenings A and C. These inconveniences were so important that they would have resulted in abandoning the use of metal ties altogether, had not Mr. Post found the way to avoid them.

The cracks of the ties, which are principally caused by the punching of the holes, are eliminated in the rolled mild steel Post ties, types X. and XI. (Fig. 2), by drilling the holes, strengthening the rail seats and rolling on the under surface two fillets to prevent the bolts from turning. The patent on this has now expired. It is my conviction that the life of these types of ties will be equal to many times the life of oak ties.

The local wear or notching of the rail base by the fastenings, which is sometimes important when spikes or screws are used on wooden ties, is considerably reduced by the use of the eccentric washer plate (Fig. 4), having a large contact surface. Thus not only is the life of the rails lengthened, but the cost of renewal of fastenings has also been reduced to less than that for the renewal of fastenings on oak ties.

If a good design of nut is used (which does not work loose by the vibration), the wear in the holes and on the rail seats of the tie, of the rail base itself and of the fastenings, is insignificant, and thereby the annual charge for renewal of this track is much less than what it is with wooden ties.

The cost of maintenance, which is somewhat higher with types I. to V. (Fig. 1) than with wooden ties, is decidedly lower with types VI. to IX., and especially with types X. and XI. and fastenings D.

The track gage keeps better on ties X. and XI. with fastenings D than on oak ties and by the solid connection between rail and tie the security is greater. As to resistance to shifting sideways and creeping lengthways by the traffic, these steel ties are far superior to wooden ties.

I would only prefer oak ties in the following limited cases: (1) On badly drained road; (2) On new high and not yet consolidated embankments; (3) On marshy ground; and, (4) With ballast which is but little permeable.

For lines with very heatvy traffic the following improvements might be made:

1. Enlarging the base of ties X. or XI. (Fig. 2) by taking 260 or 270 mm. width (instead of 235 mm.), and 2.7 m. in length (instead of 2.6 m.); this is in order to still further reduce the expenses for packing or tamping.

2. Provide the ties with two tie-plates (Fig. 4) each, in order to still further lengthen their life.

3. Increase the depth of the ends of the tie (Fig. 2) to prevent lateral displacement of the track on sharp curves and in the case of high speed or the lurching of the locomotive.

4. Use bolts 25 mm. thick, instead of 22 mm.

The supplementary expenses caused by these experiments during 17 years are very trifling and the results obtained have proved very useful. The observation of the trial tracks has enabled us to get a definite opinion as to the value of different systems of ties and fastenings. The gradual, but important improvements which we owe to Mr. Post, have contributed to assure the development of metal track all over the world, and also to ensure both greater economy and security.

Finally, I am glad to state that the result of our 17 years' work not only fully confirms the favorable opinion of many engineers, who especially studied the metal track question, particularly Messrs. Ch. Bricka, J. W. Post, A. M. Kowalski, E. E. Russell Tratman, Ch. Lebon and Dietler, but also that our results quite agree with the favorable results on some of these railways,* where this question has been examined extensively and with perseverance, by giving different systems a fair and honest trial without preconceived ideas.

*See the report of February, 1898, of the Gothard Railway, Switzerland, of which an abstract is to be found in Engineering News of April 17. It is also worthy of notice that the use of steel ties is increasing yearly on the French and German state railways, and that the percentage of steel ties on all railways of the globe taken together is steadily rising.

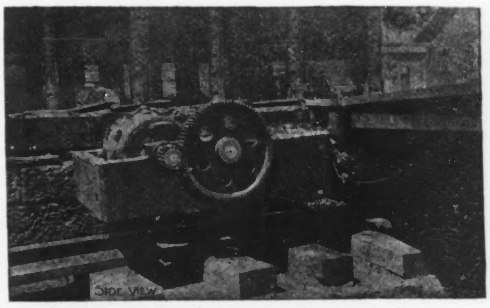
Results of Service with Steel Ties on the Netherlands State Railways, 1881 to 1898.

Trial section.	Length, ft.	Max. grade, %.	Radius of curves, ft.	Trains per day.	No. of ties.	Type of— Ties.	Type of— Fastenings.	When laid.	Observations ended Jan. 1.	Days in service.	Trains carried.	Days of 1 man's labor for maintenance— Total.	Days of 1 man's labor for maintenance— Per mile pr 10,000 trains.	No. of ties rem'v'd observation ended.
1.	3,306.24	1.20	1,640	25	1,120	Oak.	Spikes.	1881.	1895.	4,932	123,300	1,367¼	177.4	1,120
2.	3,430.88	1.20	2,460	25	1,133	I.	A	1881.	1896.	5,297	132,425	2,330¾	271	1,133
3.*	3,027.44	0.12	tan.	25	1,000	I.	A	1881.	1898.	5,966	149,150	2,652¾	311.3	305
4.	3,634.24	0.29	tan.	14	1,200	I.	A	1881.	1898.	6,044	84,616	2,078	(358)	73
5.	2,371.44	0.34	tan.	14	800	I.	A	1881.	1898.	5,966	83,524	1,489½	(391)	0
6.	1,685.92	0.16	3,280	25	600	II.	B	1882.	1898.	5,479	136,975	1,200½	275.8	263
7.	1,436.64	Level.	3,280	29	500	II.	B	1882.	1896.	4,748	137,692	1,156	308	500
8.	1,512.08	0.80	tan.	25	500	II.	B	1882.	1898.	5,479	136,975	972¾	248.4	82
9.	905.28	0.40	tan.	25	300	II.	B	1882.	1898.	5,479	136,975	737½	314.5	48
10.	3,630.96	0.39	tan.	14	1,200	II.	B	1882.	1898.	5,479	76,706	1,819¾	(34.5)	55
12.	849.52	1.30	1,640	25	300	II.	B	1883.	1898.	5,206	130,150	467¾	224.2	47
13.	1,128.32	0.65	1,640	14	400	II.	B	1883.	1898.	5,221	73,094	595½	(382.2)	159
14.	3,332.46	1.60	1,148	25	1,328	III.&IV.	A	1883.	1898.	5,206	130,150	2,527¾	308	1,328
15.	1,485.84	0.65	1,640	14	500	III.&IV.	A	1883.	1898.	5,221	73,094	780¼	(380.6)	210
16.	1,512.08	0.08	6,560	14	500	III.	A	1883.	1898.	5,054	70,756	781	(385.5)	48
17.	698.64	1.30	1,640	25	250	IV.	A	1883.	1898.	5,206	130,150	533½	309.6	77
18.	1,512.08	Level.	tan.	14	500	IV.	A	1884.	1898.	5,054	70,756	691¼	(341.9)	0
19.	1,528.48	0.08	tan.	14	505	V.	A	1884.	1898.	5,054	70,756	867¼	(424.2)	6
20.	2,220.56	0.10	6,560	14	735	VI.	C	1885-6.	1898.	4,232	59,248	636	(254.8)	0
24.	3,280.00	Level.	tan.	25	1,081	VI.	C	1887.	1898.	3,867	96,675	683¼	114.5	12
27.†	560.88	1.60	1,148	25	200	Special.	Sp'cial.	1888.	1898.	3,287	82,175	194½	222.5	0

*This section is on marshy ground.
†This section has special ties with riveted rail brace and tie-plates, Fig. 5.
Note.—The maintenance figures in parentheses are for track sections having a lighter traffic, and are therefore not directly comparable with those of the other sections. The maintenance work is low on most of these sections, but shows higher figures when divided by the number of trains.

THE WESTINGHOUSE "ELECTRIC DONKEY" FOR OPERATING TURNTABLES.

In our issue of May 27, 1897, we illustrated a very successful form of electrically operated locomotive turntable which had been put in operation a short time previously at the West Milwaukee shops of the Chicago, Milwaukee & St. Paul Ry. This installation consisted simply of an electric motor and the necessary reduction gearing mounted upon a frame provided with a single traction wheel running on the circular pit rail. Somewhat similar devices have been installed by other railways, the motor in most cases being made in the machine shops of the railway and being more or less crude in design. As these simple means have proved very successful and seem likely to be used more commonly in the future, we illustrate in the accompanying cut a special turntable motor carriage recently put on the market by the Westinghouse Electric & Manufacturing Co., of Pittsburg, Pa., and designed on the same general lines as the one used at the West Milwaukee shops of the C., M. & St. P. Ry. previously described. This motor carriage or "electric donkey," as the builders name it, weighs complete 3,700 lbs. The motor proper is of 10 HP., and the power required to operate the table when loaded with a 100-ton engine is stated to be about ½ HP. per hour.



Electric Donkey for Operating Turntables.
Westinghouse Electric & Mfg. Co., Pittsburg, Pa., Makers.

THE IMPORTS AND EXPORTS OF MANUFACTURES of iron and steel, into and from the United States, have practically reversed value conditions since 1880. From figures lately published by the Treasury Bureau of Statistics, and from a table giving the amounts for each year, we obtain the following comparison for periods of 5 years:

Year.	Imports.	Exports.	Year.	Imports.	Exports.
1880..	$71,266,699	$14,716,524	1896..	$25,338,103	$41,160,877
1885..	33,610,093	16,592,155	1897..	16,094,557	57,497,872
1890..	41,679,501	25,542,208	1898..	12,615,913	70,367,527
1895..	23,048,515	32,000,989			

The list of iron and steel exports is an elaborate one, but some of the most important items for the fiscal year, 1898, are as follows: Bar iron, 12,308,615 lbs.; cut nails, 32,310,393 lbs.; wire nails, 22,894,099 lbs.; locomotives, in value, $3,883,719; wire, 136,951,924 lbs.. Sewing machines, electrical apparatus and supplies and agricultural implements of American make practically go to all parts of the world.

FIGURES SHOWING THE CONDITION of Spain have been compiled by the Philadelphia Commercial Museum. According to this report the population is 18,000,000, of which number 8,726,519, 6,764,406 of whom are women, have no trade or profession. There are 4,033,291 men and 828,531 women engaged in agriculture. School statistics show 1,728,920 pupils with 39,582 teachers, 24,624 of whom are men. There are 97,257 public employees, 64,000 on the pension rolls; 30,477 male and 78 female physicians; 1,151 male and 32 female writers; 3,497 theatrical people; 3,497 male and 319,506 female servants; professional beggars of both sexes number 91,227; priests, 43,238, and nuns, 28,549.

A METRIC CONGRESS of the leading textile manufacturers of the world will probably be held in Paris in 1900. The congress is called mainly to overcome the difficulties arising from the want of system in numbering the thread used in the textile industry. The U. S. Consul at Liege, Mr. H. W. Gilbert, calls the attention of American manufacturers to this meeting, and says that more business would follow the employment by United States makers of weights and measures to which foreign buyers and consumers were accustomed.

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The problem of devising an economical and efficient system of generating and transmitting power for long-distance electric railway operation has become during the last few years one of particular interest to engineers. That there is likely to be a steady growth in railways of this sort there seems every reason to believe, but there is less certainty in the minds of engineers as to the best system for operating them. At present the tendency of practice appears to be to employ some form of polyphase current transmission with direct-current motors for actual car propulsion, the alternating current being transmitted to suitable sub-stations, where it is changed to direct current for supplying the motors by means of rotary converters. It is practically this system which is employed on the Chicago & Milwaukee Electric Ry., which is described elsewhere in this issue, and which is the most recent, and in some respects the most noteworthy, example of long-distance electric railway construction of which we know.

In this railway we have a generating station located approximately at the center of the road, convenient to coal and water, and equipped with modern economical engines directly connected to 5,500-volt three-phase generators. High tension transmission is used to suitably situated substations, where rotary converters change from three-phase to direct current, which by means of feeders and the trolley eventually reaches the car motors. Other combination generators supply either direct current for nearby sections or low-tension alternating current for transformation to high-pressure for transmission. The engines and generators are coupled in such manner as to permit any combination of engines and generators to avoid shut-downs. Storage batteries are used in connection with the direct-current portion of the system to balance the system, equalize the load, etc.

While this train of transformations is successful mechanically and, generally speaking, financially, and has many features which indicate that it may hold its own for some time to come, one is forced to regret that more attention has not been devoted toward developing a practical two or three-phase street railway motor. There seems to be no inherent reason why such motors cannot be made to meet the rigid requirements of power, speed, control and efficiency as well as the present direct-current motor has. The matter of conveying the current to the moving car or train would entail more apparatus, it is true, but, on the other hand, the extreme simplicity and the smaller amount of attention required at the central and sub-stations would seem more than to counterbalance the double trolley necessary. In such a system the current could be generated at any desired voltage up to about 10,000, using stationary armature machines. At the sub-stations static transformers would step this pressure down and feed directly into the trolley circuits, thus eliminating the rotary transformers and the present annoying factors of electrolysis, the effect upon telegraph lines and ground return telephones, etc.

Of course, it can be argued that there is nothing to be gained by the less complicated system, and that the use of the rotary converter and storage batteries, while they increase the amount of apparatus, and cause a loss in transformation, they more than compensate for this by permitting the installation of machinery of smaller output and by insuring better regulation. It is, however, quite possible to use a storage battery plant in connection with a three-phase generating system, if instead of using an ordinary three-phase generator, motor generators are employed, as in the plant already mentioned, in which the dynamos are directly connected to the engines and can run either as direct-current generators, for feeding circuits near the station, or as three-phase generators, for distant circuits. To illustrate the statement just made, suppose such a plant, equipped with an adequate storage battery plant, with the direct-current side connected with the battery plant, and the three-phase side feeding the line during the times when the load was light, as in the early morning, midday or late at night. During these hours of light traffic the engines could be operated at an economical load charging the battery. As the load on the three-phase circuits began to increase, however, the engines could be maintained at this proper load, the battery voltage increased by suitable combining switches, and the direct-current side of the generator used as a motor to help over the "peak." In this way all apparatus could be run at full load, which would thus permit the use of smaller engines and generators, because they would always be working at their most economical point.

Not a little attention has been paid of late in various assemblies of engineers to the standing of engineering as a profession; and it is encouraging to note, despite isolated discouraging instances, that the general public is slowly coming to understand that engineering is a profession, governed by the same rules of honor that govern the legal or medical professions, and its members have a right to similar honorable treatment.

There is, however, much hard work still to be done by the profession in securing the position that it deserves. Perhaps one of the hardest things an engineer who stands up for the honor of the profession has to overcome is the attempt of occasional manufacturing concerns to undermine his employers' confidence in his integrity and impartiality.

An aggravated instance of this sort has just come to our notice, where a manufacturing concern added insult to attempted injury by tendering a bribe to the engineer. Circumstances made this concern especially anxious to sell its goods to this particular customer, as he had already made extensive use of them, but not with perfect satisfaction. This much is said in order to present any extenuating circumstances in the case. The story, as briefly as possible, is as follows: An engineer was called in to make plans for a certain work. He found a patented apparatus in use which had become inadequate for its purpose, through increase in the output of the plant, and the efficiency of which, as compared with other methods, was at least open to question. For a variety of reasons the engineer recommended the adoption of a new system, under which contracts could be let in the open market. The client accepted his recommendation, and work by day's labor was started, although the letting of contracts for the bulk of the work was postponed until warmer weather. About this time, the company whose apparatus was to be displaced went to the client and urged a reconsideration. Later the engineer received a letter from one of the officers of the company in which it was assumed that the engineer's recommendation of the non-patented process was because his services would be more largely required in its installation, and an offer was made of 5 per cent. commission, if he would alter his recommendation and favor the company's apparatus. The offer was, of course, refused.

Next the company began a stubborn fight to discredit the engineer in the eyes of the client, taking the ground that the engineer in advising the adoption of a certain process was a trade competitor, and entitled to no more consideration than themselves. They brought forward the old argument that they could furnish undoubted financial guarantees as to the cost of their plant, which the engineer was, of course, unable to do. They unblushingly admitted their offer of a commission to the engineer, and declared that the offer and acceptace of such commission was a matter of course.

We are exceedingly pleased to state that, notwithstanding all the specious arguments and powerful influences which the company's agents brought to bear, the client was wise enough to perceive their fallacy and to order the work carried out on the engineer's original plans.

We doubt not that not a few of our readers have had similar experiences at one time or another; and have felt the need that in some way the profession should stand together to repel such assaults as this on its integrity and honor.

It is one of the hardest tasks that comes to many an engineer to stand up before a Board of Directors or City Council, to defend his own integrity, and make plain that he stands as their impartial adviser and representative, while the wealthy and influential promoter, the glib-tongued lawyer or the ubiquitous sales agent, who is urging that the engineer's advice be rejected, is merely working for his own profit. Is there any way in which engineers can uphold their professional standard and assist each other to win the day in hard-fought battles of this sort? We know of no way except for each man to stand up manfully and fight his own battles to the best of his ability; and further, to see that so far as possible no professional recognition or distinction shall attach to the man who degrades the honor of the profession by accepting commissions from those whose wares he recommends, or who undertakes to give advice which he is not competent to furnish.

If the engineering profession can do this, honest manufacturers will have no reason, as it must be be confessed they now sometimes have, for offering commissions in order to secure a hearing for their goods or for appealing from an engineer to his principal because their goods have been judged unintelligently or unfairly.

NATIONAL SOCIETIES OF MUNICIPAL OFFICIALS.

The organization of the League of American Municipalities, a year ago, naturally raised the question, is another society for municipal officers needed? The recent meeting of the League at Detroit, reported in our issue of Aug. 11, affords some answer to the question, but does not settle it. Doubtless most of those in attendance at Detroit felt well repaid for going there and for having organized the League, but in the long run would not more have been gained if the members of the new society had joined some of the existing associations, thus strengthening them and widening their field of usefulness? There were already in existence a number of societies, some limited to one branch of municipal activity, and others broader in scope, which would have given warm welcome to all who have united with the League. Whether these other societies would have met all the wants of those who have joined the new one is a fair subject for discussion. The question

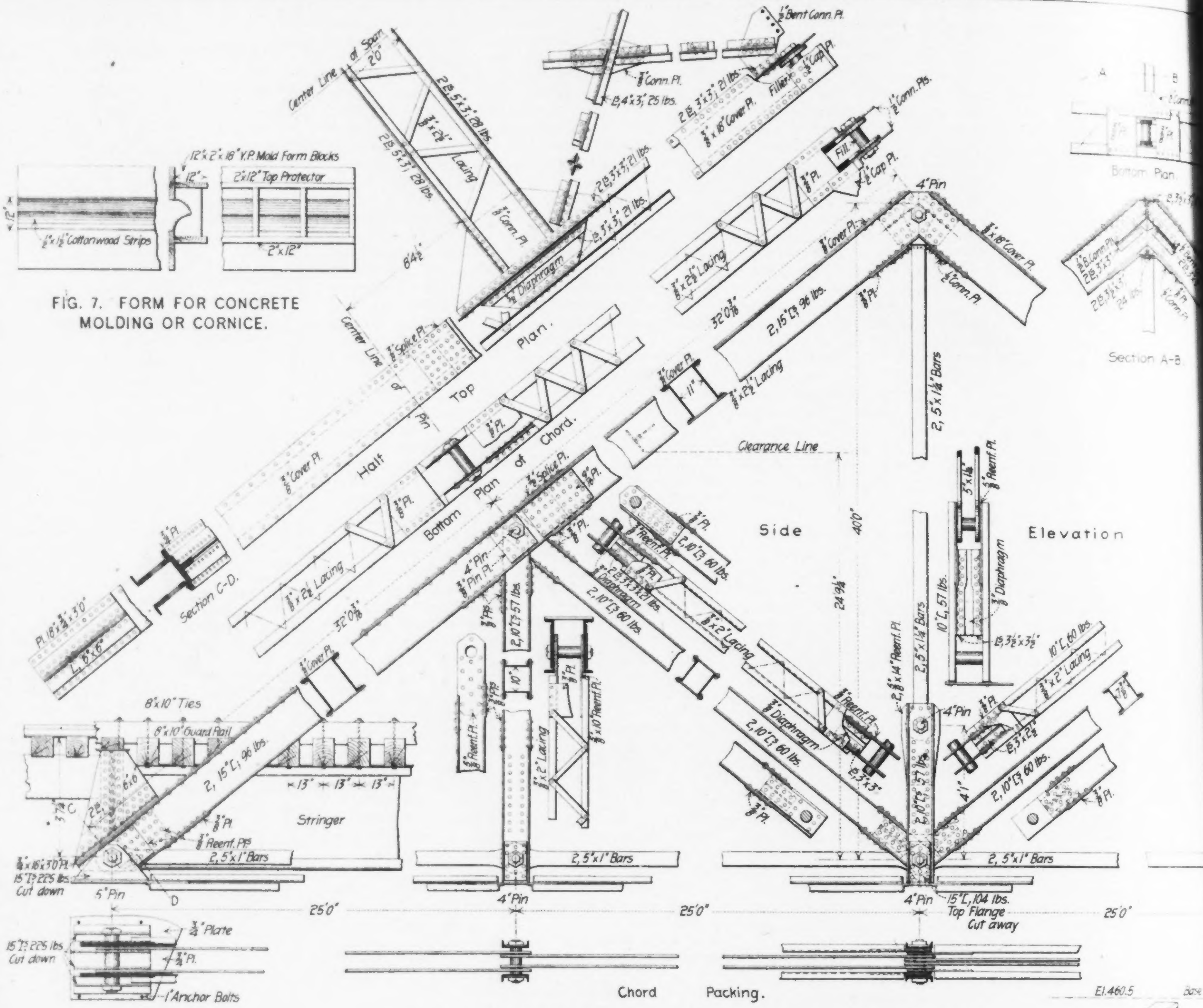

FIG. 7. FORM FOR CONCRETE MOLDING OR CORNICE.

FIG. 2. DETAILS OF "A" TYPE OF THROUGH TRUSS OF 100 FT. SPAN.

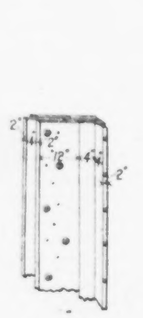

FIG. 8. WAKEFIELD SHEET PILING (SHOWING CORNER PILE.)

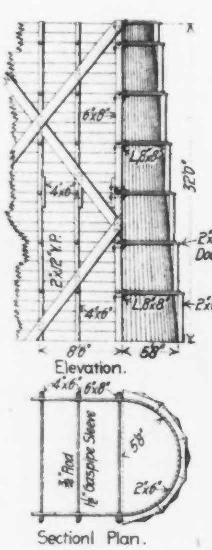

FIG. 6. MOLD OR FORM FOR CONCRETE PIER.

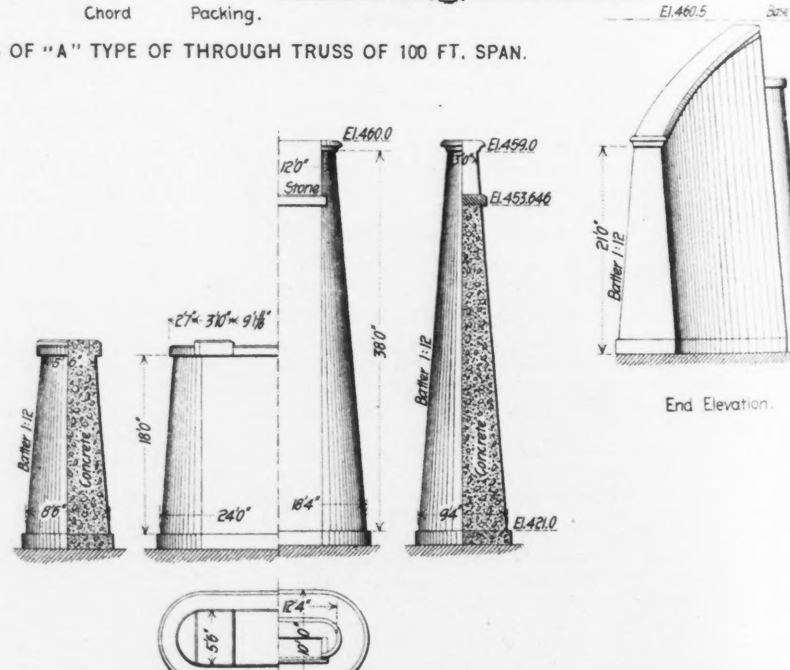

FIG. 5. DETAILS O

STEEL BRIDGES AND CONCRETE PIERS; KANSAS

Robert Gillham, M. Am. Soc. C. E., M. Inst. C. E., Chief Engineer and General Manager.

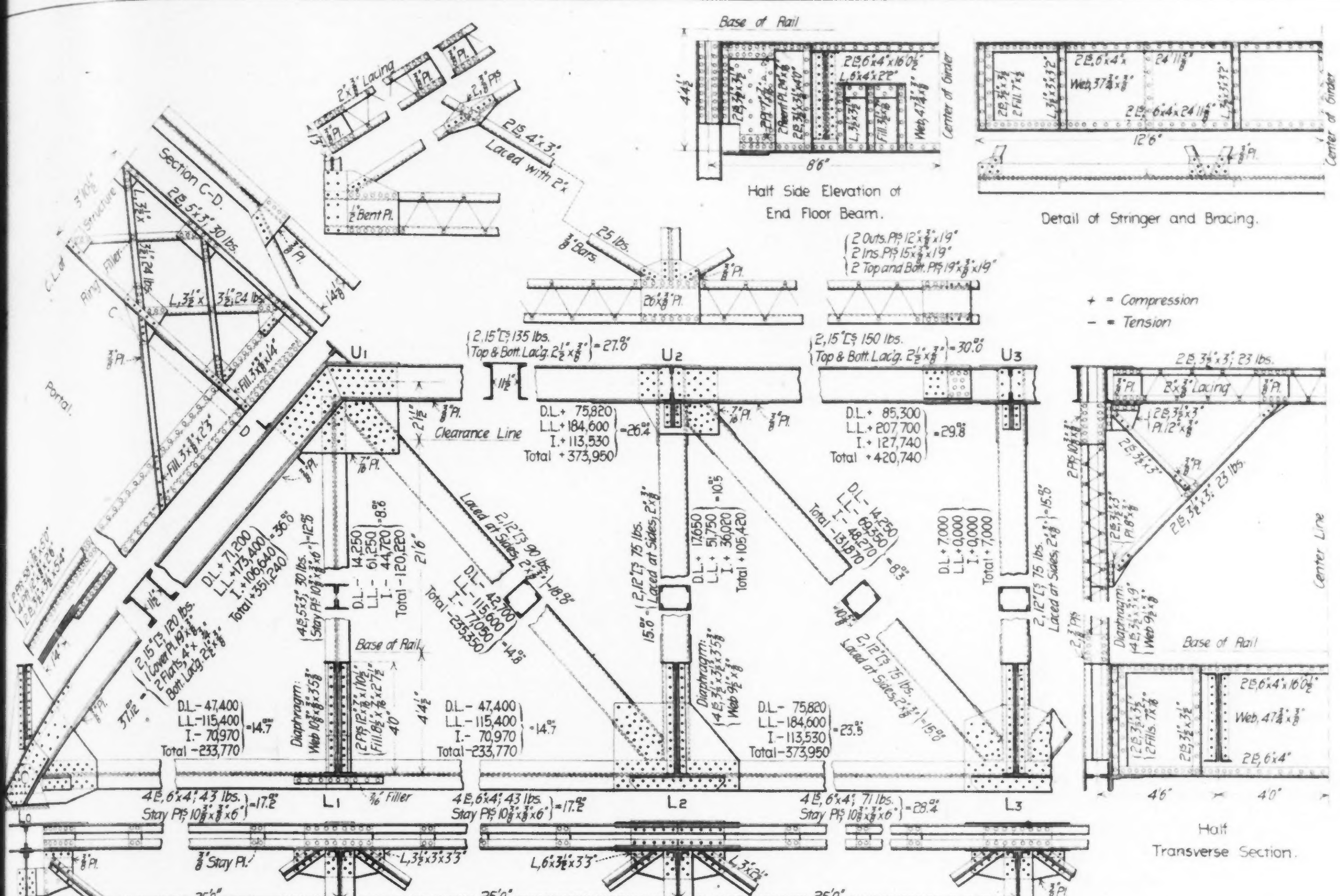

FIG. 3. RIVETED THROUGH TRUSS OF 150 FT. SPAN.

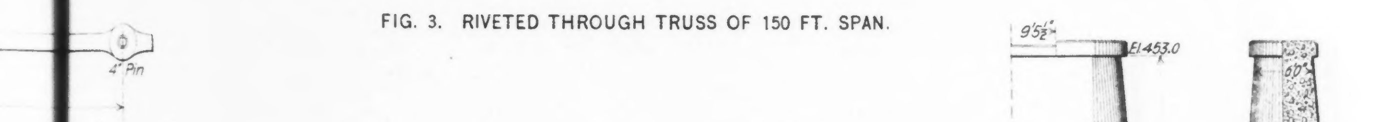

FIG. 5. DETAILS OF CONCRETE PIERS AND ABUTMENTS.

IERS; KANSAS CITY, PITTSBURG & GULF R. R.

ger. J. A. L. Waddell, M. Am. Soc. C. E., Consulting Bridge Engineer.

CHAS. HART & SONS, LITH., 80 VESEY ST., N. Y.

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opens a broader field of inquiry and gives occasion to pass in review the causes which led to the formation of the various engineering and other societies open to the municipal officials of the United States and Canada, and to consider the extent to which they are meeting the needs that brought them into existence, or the opportunities that have since arisen. We shall not undertake an elaborate analysis of the subject, our primary object being to examine the claims of some of the newer societies to membership and to indicate how all may be improved in some essential particulars.

When we consider the rapidity of the growth of American cities, the diverse elements of their population, physical conditions and local industries, and perhaps above all the great lack of uniformity of their governmental organization, due to the fact that each state legislature frames city charters not only without regard to other states but in many cases with equal disregard to uniformity within its own limits, it is a wonder that our municipal affairs are not worse than they are. Growth without system and public improvements without knowledge of what has been done elsewhere have been the two great characteristics of American cities. The current of municipal life has been so rapid and our citizens have been so engrossed in their own affairs and in the success of partisan national government that they have been carried along by the stream instead of controlling it. The branches of municipal government that show the greatest number of exceptions to this rule are the very ones that have longest had access to membership in technical societies of national scope, namely, city engineers, water-works superintendents and health officials.

Not so very many years ago the American Society of Civil Engineers was the only national body of men concerning itself with the technical aspects of municipal affairs. That society, ever since it began to be a force in the profession, has given a portion of its attention to water supply, sewerage and paving, but it covers none of these subjects thoroughly and has strangely neglected so important a matter as garbage disposal. Of the many things the city engineer is called upon to deal with, bridge engineering is perhaps the only one whose progress and status is shown by the transactions of the society.

In the earlier days of the American Society of Civil Engineers the consideration it gave to sewerage and paving was more in proportion than now to the development of those branches, for such work was small in amount and done in a haphazard manner, except in some of the larger and more enterprising cities, where it was in the hands of engineers to whom the society was open and capable of meeting most of their wants. With water-works officials the case was different. They were already a large class, with a great variety of perplexing problems to solve and much valuable information to exchange with their fellows, when the society was first coming into prominence. The society met practically none of the wants of the water-works men who were not engineers and only a small part of those who were, largely because those wants were outside the scope of the society. In the early eighties, therefore, the American and the New England Water-Works Associations were formed, and have since grown into two strong bodies, with many members who are engineers, but far more who are not. These associations have the great advantage of being able to give their whole energies to water-works affairs, going into as much detail and personal experience as the members may find profitable. The New England Association holds monthly meetings through the winter and publishes quarterly proceedings of much value, thus taking a high rank among technical societies. The American Association, though holding only one meeting a year, does good work. Taken together, these societies seem to meet all the wants of water-works officials, and if they lack in consideration of questions of pure hydraulics this deficiency is made good by the American Society of Civil Engineers.

Chronologically, there should be mentioned as having been formed about this time the American Public Health Association, a body which has broadened its practical scope and usefulness until it now includes within its membership a few engineers, many health officers, both city and state, and some laymen. The medical profession, naturally enough, has been the dominant element in this association, but such engineering questions as water and sewage purification, street cleaning and garbage disposal figure largely and well in its proceedings. The municipal health officer has for years had his needs well provided for here.

Other phases of municipal affairs provided for by national associations which may be mentioned but need not be discussed, are education, libraries, charities and correction, and quite recently parks and cemeteries. Street railways, gas and electric lighting plants are wholly or largely under private ownership, and have their own national societies, so there are no men devoted to these industries without a society haven.

Gradually, for some years prior to 1894, there was felt to be a need for an organization which would bring together those municipal officials responsible for streets, sewers, and some other public works only lightly touched upon, if at all, by the existing national societies. It seemed especially desirable to reach a class of city officials more uncertain, perhaps, in their tenure of office, than city engineers and water-works superintendents, and therefore all the more in need of the instruction and stimulus of a live national society. As it was realized that many of the individuals joining such a society would soon drop out, it was decided to allow cities to join in their corporate capacity, with the privilege of individual membership for as many officials from each city as might desire it. Along somewhat such lines as these the American Society of Municipal Improvements has developed during the last four years. Some of the officials designed to be reached by this society are more or less closely in touch with several branches of municipal affairs, such as boards of public works, while mayors, councilmen and financial officers are or should be interested in all classes of public improvements and certainly need instruction from some quarter, coming as so many of them do, from business life to municipal affairs and dropping out again before they gain much experience in their new field. With a membership so diverse in its official duties and interests it is not strange that the new society has taken up practically the whole range of public improvements, with some financial and administrative questions in addition.

Within the past few years it has begun to be recognized that the machinery of city government needed attention and that good men were as essential as good methods. Accordingly, good government clubs and municipal reform associations have sprung up all over the country and the National Municipal League has been organized to bind together these clubs and such individuals as desired to further the cause of good city government. This national organization has by no means been confined to municipal officials, but it has afforded an opportunity for the co-operation of all such officials as are interested in its objects.

With all the societies described above it would certainly seem as though no further national organizations of municipal officials were needed. But in the summer of 1897 a call was issued for a convention of mayors and councilmen, which resulted in the formation of the League of American Municipalities. It is hard to say just what causes led to this step. It seems likely that not all the officials responsibble for it were aware of the existence of the various societies already open to them, while others may have felt the need of a comprehensive society, less technical in its character than those named. The attention given by the new society to questions of municipal ownership suggests the idea that there was a desire for a free and extensive interchange of opinion on this subject. Judging from the report of the Detroit meeting of the League (Engineering News, Aug. 11, 1898), mayors predominate among its most active members. The discussion of technical subjects at the Detroit meeting seemed to partake largely of the relation of personal experiences, often limited in time and extent. All this is good so far as it goes, and if combined with more papers and discussions from men who were thorough masters of their subjects it would be all that could be desired. The danger seems to be that, in certain lines, we shall see at these meetings blind leaders of the blind, unless a new element is infused into them.

The League of American Municipalities held its first meeting so closely in advance of the fourth meeting of the American Society for Municipal Improvements as to deprive the latter body of some of its intended speakers. This, coupled with the similarity of the two bodies in scope, has suggested the idea of deliberate rivalry on the part of the new organization. That the two societies appear to be serious rivals there can be no question, but whether this was intended by the new one we cannot say. There are indications that the two societies are drifting into quite different lines, both in membership and methods. When the League was being organized it seemed very doubtful whether it was needed and its future must settle that question. Could the two bodies work together it seems as though material gain would result. The mayors and councilmen would learn from their fellows more actively engaged in the design and construction of engineering works, and the engineers and various commissioners responsible for such works might be benefited from a more intimate association with other classes of city officials, not the least of which benefit would arise from a growing respect for technical men on the part of the other officials. A mere reduction in the number of societies demanding time and money would be an advantage which needs only to be mentioned to be appreciated by many.

Before closing this article it seems desirable to call attention to some of the limitations and possible improvements in the societies passed in review. First of all, it is to be noted that a national society holding only one meeting a year and publishing merely an annual report must of necessity have a somewhat narrow range of usefulness, especially where there are many and rapid changes in membership. More frequent meetings may perhaps be held only with difficulty, or if held can be attended by only a small part of the membership, but they add life to an organization. Committee meetings might be practicable at intervals during the year where larger gatherings are out of the question. The conclusions reached at such meetings, together with other information, might be printed and sent to all members at intervals between the publication of the annual reports.

Many society meetings lose in value because of too crowded programmes. This requires haste in the reading of papers and often limits or entirely cuts off discussion.

The discussions brought out by many papers in the most successful societies, notably in the Institution of Civil Engineers, are of greater value than the papers themselves, and with the papers often form a resume of the whole subject in hand. Most of our societies are lame in this respect. Written discussions should be encouraged, but neither these nor much oral discussion can be expected on papers at all technical in character unless the papers are printed and sent to members in advance of the meetings, a practice that most of the societies reviewed above do not follow.

If discussions are to be of value those participating must be held to the subject by the chairman of the meeting, and in addition the discussions should be edited before publication, making free use of the blue pencil if desirable.

While the officers of any society can do much towards the securing of good papers, these are often largely the result of volunteer efforts on the part of the members, a fact that should be remembered and acted on by all.

In conclusion, we think the limitations and possibilities indicated above show that it is more desirable to strengthen on old society, where possible, than to create a new one. Good work requires good men and plenty of money, and these are best secured by uniting rather than dividing forces.

LETTERS TO THE EDITOR.

Errata in "De Pontibus."

Sir: Permit me through your columns to call the attention of those of your readers who have purchased my "De Pontibus" to a few errors therein. On page 203, 19th line, for "is" write "are." On page 204 the formula should read:

$$P = 550 \frac{HP.}{v}$$

The 550 factor was dropped in typewriting the chapter, which, by the way, was written three times to bring it into its final shape. The other chapters were all rewritten once, and in some cases twice. The first copies of the book that were issued contained also the following typographical errors. On p. 141 the heading should be Chapter XIV., not Chapter XVI., and on p. 157, line 22, last word should read "rever-," the "ver" was left out. These are all the errors thus far discovered, barring blurred letters and omitted hyphens. It was my endeavor to bring out this work without a single error of any kind, but it seems that I was unsuccessful, notwithstanding the combined efforts of my office assistants, several friends and myself. It is by no means an easy matter to produce a book containing some 160,000 words without letting a few errors creep in. It is possible that there are still others than those here indicated. If anyone discovers any, I hope that he will do me the favor to point them out without delay. Very respectfully yours,

J. A. L. Waddell.

Kansas City, Mo., Aug. 13, 1898.

Power of Windmills.

Sir: Mr. Fargusson's suggestion of the formula HP. = $.001 D^3$ as representing the power of a wind engine in a 16-mile wind seems to fit some of the results found by Mr. Murphy as published in Water Supply Papers, No. 8, but comes far from representing the results found by experiments of Mr. F. H. King, of the University of Wisconsin. Mr. Fargusson considers the Aermotor favorably and gives the formula HP. = $.002D + .0001D^3$ as his judgment of its power. This would rate a 16-ft. Aermotor at 0.44 HP. Mr. King found it to give 1.08 brake HP., and about 0.25 actual HP. Mr. Murphy's 16-ft. wheel with which I am familiar gave 0.433 HP., which is very close to Mr. Fargusson's formula. The term HP. as applied to a windmill is apt to be very misleading, as its power at any wind velocity is a function of its load as well as of the wind energy at its disposal. This is clearly shown by comparing the two results cited. Since during only about 4% of the year is the wind at 16 miles per hour, and only about 10% even in the immediate neighborhood of that rate, the usefulness of a windmill rating based on a single wind velocity and unknown load is doubtful. For instance, knowing that a mill actually gives 0.5 HP. in a 16-mile wind, could one predict what would be the output in any season? I think not. This is usually desirable to know, and only by knowing the wind distribution can it be discovered. The following figures which I am using are based upon records kept for seven years, and represent mean values for a well loaded mill reaching its maximum efficiency in a 16 to 20-mile wind:

Table Showing Millions of Foot Pounds of Output per Square Foot of Sail Area in Each Month; Distribution Based on Kansas Wind Records:

January	1.38	July	1.17
February	1.6	August	.8
March	2.58	September	1.59
April	2.49	October	1.6
May	2.07	November	1.9
June	1.58	December	1.67

The figures represent millions of foot pounds of work in lifting water, done by each square foot of sail area exposed by the mill. The figures include the effect of winds between 6 and 30 miles per hour. They also include the varying efficiency of a typical mill between these extremes of velocity and distribution. They cannot take into account the very important factor of relative load on the mill, but they do take into consideration the wind distribution and the variations in efficiency of a typical plant at the various speeds. To estimate the probable amount of water pumped by a 10-ft. mill, say in July and August, add the figures from the table for those months 1.17 + 0.8 = 1.97, representing 1,970,000 ft. lbs. of useful work for each square foot of sail area. A 10-ft. wheel has about 55 sq. ft. of sail area, giving a total for the two months of about 100,000,000 foot pounds of useful work. From this the quantity from any depth of well can be readily found. I know of no other figures or formulas which will give this kind of information. From the nature of the problem such estimates cannot be very accurate, yet, using these figures which do not include Mr. King's work in any way, they check with his work so closely as to be remarkable. They also check with some of Mr. Murphy's results as far as the data is discoverable. Mr. King's 16-ft. windmill and pumps actually did 2,679,000 foot pounds of useful work in the year. The sail area was about 135 sq. ft. Using the figures as I have deduced them would have predicted a yearly output of 2,700,000 foot pounds.

Very respectfully,

O. P. Hood.

Houghton, Mich., Aug. 13, 1898.

BREAKWATER CONSTRUCTION ON THE AMERICAN COAST.*

In this paper reference will be confined to that class of breakwater construction which belongs to the ocean coast of our country, thereby omitting any detailed allusion to breakwater construction upon the great lakes, where the conditions surrounding the problem are so widely different from those belonging to the ocean coast, that but little correlation exists between them.

Without a definition there might be some misunderstanding as to the class of constructions to which reference will be made, since we find that the terms breakwaters, jetties and piers are sometimes confusingly used. A breakwater is an artificial structure, providing a harbor or roadstead with protection against waves, and differs from the other constructions named, in that it is essentially a wave-breaker.

Breakwaters, through their object, may be divided into two classes, viz., those which give shelter and protection to commercial harbors or their entrances; and those sheltering an anchorage or roadstead: the latter are denominated harbors of refuge, and are only used by vessels in transit, which have occasion to escape from the violence of passing storms. Each of these classes might be technically further divided according to their particular type of construction; which in turn is decided by the question of initial or ultimate economy, with due regard to effectiveness. For example, where durable rubble stone could be cheaply obtained the breakwater would probably consist of random stone, of suitable sizes, deposited in the form of an embankment, or mound. Where such stone could only be obtained at considerable cost, concrete blocks might be substituted, and either deposited as the random stone would be, or in the form of a massive and regular wall; or the stone might be used for a substructure surmounted by the concrete blocks as a superstructure. The latter form has been extensively used abroad; but has never been adopted, though frequently considered, upon the ocean coast of our country.

An abundance of durable stone, convenient to the coast, is to be found along that part of the Atlantic north of Cape Hatteras, and along the Pacific coast. This fact, supplemented by the modern methods of cheap quarrying and handling of stone, has made its use much more economical than that of concrete blocks, and has thereby decided, in this country at least, the question in favor of random stone for breakwater construction along our ocean coast.

The limits of this paper will not permit more than a passing reference to the forces impressed upon breakwaters through wave action; but brief allusion thereto becomes necessary, since the amount and direction of these forces must be carefully considered before the proper design and position of the breakwater can be adopted, or the general principles of breakwater construction understood. In the problems of the engineer, the maximum force likely to be impressed upon the structure determines the necessary stability to be given the construction. While it is difficult to estimate, except approximately, the force or impact of breaking waves, observation and experience furnish some data for deductions. The English engineer, Thomas Stevenson, followed by others, has furnished the results of experiments, through the aid of marine dynamometers, which assign values to the wave forces to be met and resisted. These observations recorded pressures in the North Atlantic of 6,000 lbs. per sq. ft.; and in the German Ocean of 3,000 lbs. M. Le Ferme, from the destruction of the beacon at the mouth of the Loire, calculated that the wave forces impressed upon the work must have exceeded 4,800 lbs., and probably approximated 6,000 lbs. per sq. ft. The most remarkable observed instance of the power of waves was afforded by the movement of a solid mass of masonry, set in cement in the form of a massive block, weighing 1,350 tons, at the end of the Wick-Bay breakwater. The mass, 45 ft. wide, 26 ft. long and 21 ft. high, was completely turned around upon its base, and at last tilted off its foundation.* Extraordinary as this may appear, it was subsequently surpassed when another concrete block, substituted for the one just described, was in like manner carried away, though it contained 1,500 cu. yds. of concrete masonry, and weighed about 2,600 tons.

In 1884, Mr. William P. Judson, C. E., made some observations upon Lake Ontario at Oswego, N. Y., to determine the height, velocity and impact of the waves upon the breakwater at that locality, with the following results: During a severe northwest gale, the waves at a distance of 1,000 ft. outside the breakwater, attained a height of from 14 to 18 ft. above the normal surface of the lake, with a velocity of from 30 to 40 miles per hour. Dynamometers attached to the face of the breakwater at depths of 8 ft. below the surface, at the surface, and 8 ft. above the surface, recorded pressures, respectively, of less than 10 lbs., 600 lbs., and at the higher elevation about 1,000 lbs. per sq. ft. The instances referred to are the maximum forces which have been observed, but, since they depend upon local conditions, cannot be assumed as those which are to be the standard in all cases. The height and force of impinging waves at any locality depend upon the force and duration of winds, the depth of the water, the fetch or distance over which the waves move, and the angle of incidence at which they strike the breakwater.

Theoretical conditions indicate that the energy of the impact of waves varies as the cube of the waves' height. Stevenson, from observation, developed the empirical formula, $h = 1.5 \sqrt{d}$, for the determination of wave-heights, which is considered as representing with tolerable accuracy the heights of waves during heavy gales, in which h is height in feet, and d the fetch of the waves in miles.* The results of this formula for low values of d are somewhat too small, while for large values of d they are too great. Scott Russell considers that 27 ft. is the greatest height of waves in the British seas; and Sir G. Airy, from theoretical consideration, concludes that from 30 to 40 ft. would be the extreme height of unbroken waves, except under rare conditions. Dr. Scoresby observed waves in the mid-Atlantic which averaged 26 ft. in height, and moving with a velocity of over 30 miles per hour, while the highest wave he estimated to have a height of 43 ft. Waves have been observed off Cape Horn with a height of about 50 ft. Previous to the laying of the first Atlantic cable, an effort was made to carefully collate the most reliable information upon this point; the result was that 50 ft. was considered as the extreme height of waves in the open sea.

On the assumption that, at the instant of breaking, the vertical section of a wave at the surface is a common cycloid, and that the height, d, of the wave is not greater that the depth of water in which it moves, theoretical

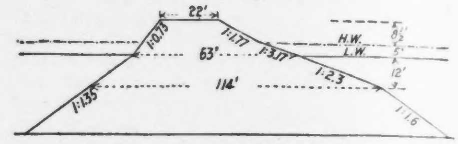


Fig. 1.—Section of Old Delaware Breakwater.

considerations indicate that the maximum wave energy, in foot-pounds per lineal foot of the wave crest, =

$$\frac{64 \pi d^3}{32} = 2 \pi d^3;$$

the weight of a cubic foot of sea water being taken at 64 lbs.† for wave-depths of 10 ft., this gives an energy of 6,300 foot-pounds, and, for wave-depths of 15 ft., about 21,000 foot-pounds. If the form of the wave at the moment of breaking is that of a prolate cycloid, as some authorities assert, its energy would be about 60% greater than that above stated. While such calculations are more curious than valuable, they nevertheless serve to convey some idea of the forces impressed upon breakwaters by wave-action.

The destructive effect of waves of translation also depends greatly upon the angle of incidence between their line of impact and the axis of the breakwater, and, other conditions being equal, this force of impact will vary with the sine-square of the angle of incidence. At Wick it was found that by changing the direction of the axis of a part of the work, so as to reduce this angle from 90° to about 80°, the breakwater withstood the action of waves, which previously had been such as to repeatedly destroy the work.

From this brief consideration of the wave-forces impressed upon a breakwater, it is evident that its design, from the standpoint of stability, must depend upon conditions arising from the location of the work. In the case of random-stone breakwaters, this question of stability depends upon the cross-section of the work, and the dimension of the stone used upon the active slopes. Such a breakwater essentially consists of two parts, viz., the substructure or that part below the surface of the water, and the superstructure or that above water. The wave-action which it is necessary to arrest by a breakwater does not extend to a great depth below the surface of the water; therefore, the superstructure and the upper part of the substructure are the effective parts of the breakwater, as well as the parts of the construction which are subjected to the most destructive action of the waves. A large part of the substructure, therefore, becomes simply a means of supporting the superstructure, and the limited part of the substructure which is exposed to marked wave-action. In general terms, the necessary dimensions of the superstructure and the upper part of the substructure, determine the dimensions of the remainder of the work. While the superstructure must resist severe wave-action, it must likewise preserve the needed tranquility of the areas behind it, by being carried to a height sufficient to prevent waves being thrown unduly over the top of the structure.

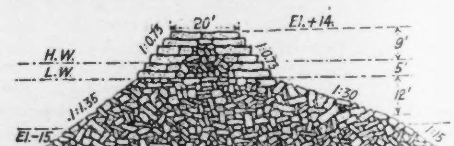


Fig. 2.—Section of New Delaware Breakwater.

The bounding surfaces of a random-stone breakwater consist approximately of plane surfaces, forming the top, bottom and sides: the latter, at more or less of an inclination dependent upon the special part of the work to

*From Proceedings of the Engineers' Club of Philadelphia; paper by Louis Y. Schermerhorn, M. Am. Soc. C. E.

*"Harbor and Docks," Harcourt, pp. 28, 31.

*"The Design and Construction of Harbors," T. Stevenson, p. 25.

†"Report, Chief of Engineers, 1893," p. 1321.

which they belong, are called the slopes. One side of the work is exposed to the action of the sea, while the opposite side rests in the comparatively quiet water of the protected area. The sea face of the work, while receiving and arresting the energy of the waves thrown upon it, is not, from the limited depth of wave-action, exposed throughout all of its depth to the same amount of destructive force; therefore that side of the breakwater need not be provided, for its entire depth, with the same measure of stability.

Experience upon American breakwaters has determined that the depth to which energetic wave-action extends is about 12 or 15 ft. below the surface of water; and this depth is generally assumed, upon our works, as the approximate plane of rest for the material which is used.

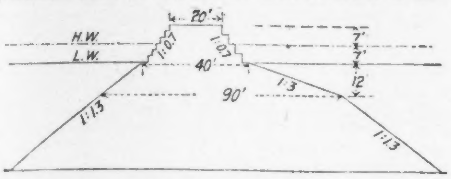


Fig. 3.—Section of Proposed Breakwater, San Pedro, Cal.

Since our breakwaters are all placed in tidal waters the position of this plane must vary with the amplitude of the tide, which in the breakwater locations to be referred to is between 3 and 8 ft. Upon leading European breakwaters, of nearly the same type as those under consideration, the increased exposure of their sites, and the height of ocean waves impressed upon the works, lowers the plane of rest to a depth of about 20 ft. below the surface of the water; while the tidal range of their localities is nearly double that above given for American breakwaters. It is evident that these facts must materially change the details of American and European practice.

In a qualified sense, the so-called plane of rest is more of an abstraction than a well-observed and established fact, and it should not be inferred, from the use which is made of this term, that a clearly defined depth-limit to the disturbing effect of wave-action can he undeniably assumed, within the limits usually ascribed to this plane. It is true that a point must exist in a wave's depth where the impulsive force is a maximum, and it is equally true that upwards and downwards from this point the force must decrease; but the law of this decrease does not permit the conclusion that wave-action entirely ceases at the depths assigned to the plane of rest in breakwater construction. It simply means that at the assumed depths the disturbing effect of the waves has been so reduced that it is no longer able to move the special stones constituting the slopes at such depths. If these stones were larger the plane of rest would be raised, and conversely, if they were smaller it would be lowered.

From the foregoing consideration the slopes of a random stone breakwater may be divided, from the forces impressed upon them, into the following: (1) that part of the sea face which is below the plane of rest; (2) that part which is between the plane of rest and the surface of the water; and (3) the part above the surface of the water. On the harbor side but two divisions of the slope need consideration, viz., that below and that above the water surface. Upon the sea face below the plane of rest the random stone will assume a slope coincident with the angle of repose of the material, unaffected by wave-action; on that part between the plane of rest and the surface of the water, the slope will be formed by wave-action, modified somewhat by the dimensions of the stone constituting the slope; while above the surface of the water, both on the sea and harbor faces of the work, the slopes will be artificial, and dependent upon the size of the stone and the care with which they are laid. On the

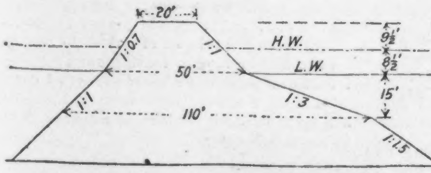


Fig. 4.—Section of Sandy Bay Breakwater.

harbor face, where the structure is not exposed to wave-action, the slope from the surface of the water to the bottom will be that of the angle of repose of the material. These several slopes, as determined by experience and practice, will be considered later.

Where the wave impinges upon a nearly vertical surface, the wave is thrown to a considerable height, and in falling is reflected, in part, seaward; while experience indicates that a talus causes the wave to break and thereby dissipate its energy upon the sloping surface on which it impinges; this is said to be fully realized when such a slope is about 1 on 3. Upon the vertical face type of breakwaters waves have been observed as thrown to a height of 200 ft.; this fact serves to strikingly illustrate the energy contained in waves, and the stresses brought upon the construction in the arrest of wave-force. Upon any slope less than the angle of repose of the material, the size of the stone determines largely the stability of the slope to resist forces tending to disturb its equilibrium. In general terms such stability increases with the dimensions of the stone used; and, within proper limits, the active slopes of the breakwater must be increased—with a proportional reduction of the aggregate volumes of material used in the work and a consequent reduction in cost—by the use of larger individual masses of stone in these slopes. The limit of such economy is reached, however, when the increased cost of larger stone equals the saving in the cost of the aggregate volume of stone used in the work. This has led in European practice to the use of immense blocks of concrete upon the most exposed part of the sea slopes of random-stone breakwaters.

The dimensions of the stone used in the active slopes of the substructure usually vary between 2 and 5 tons, with smaller stone constituting the interior volume of the work. Since the superstructure is above the surface of the water it permits of the cheap application of labor to the orderly arrangement of the stone constituting its mass, whereby a saving in volume, and consequent cost, is obtained by using very large stones, laid with some approximation into the form of a rough, though strong and heavy wall. Upon this part of the work stones weighing from 3 to 10 tons each are used on the sea and harbor faces, with a core of somewhat smaller stones.

Under American experience and practice the general slopes of the several parts of the sea and harbor faces are as follows:

Sea face, below the depth of 12 to 15 ft...1 on 1 to 1 on 1.5
From 12 to 15 ft. depth to low water..1 on 3
Low water to top of superstructure...1 on 0.7 to 1 on 1
Harbor face, from bottom to low water...1 on 1 to 1 on 1.3
Low water to top of superstructure...1 on 0.7 to 1 on 1

The superstructure usually has a width on top of about 20 ft., and with the slopes above given the dimensions of the resulting sross-section of the entire structure become dependent upon the height of the superstructure and the depth of water in which the breakwater is placed. Upon the two most recent works, viz., the breakwater for the National Harbor of Refuge in Delaware Bay, which is now in progress, and the proposed breakwater for the Harbor of Refuge on the coast of Southern California, at San Pedro, the top of the superstructure is placed at a height of 14 ft. above the plane of low water. With these

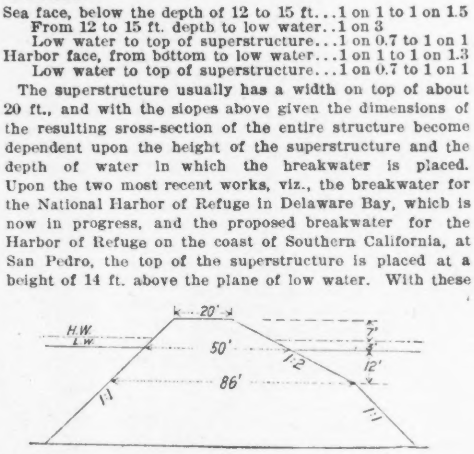


Fig. 5.—Section of Point Judith Breakwater.

dimensions for the superstructure and the slopes above given, the breakwaters at the localities named obtain a width of about 40 ft. at the plane of mean low water, and about 90 ft. at the assumed plane of rest, 12 ft. below the plane of low water.

The most typical random-stone breakwaters of the ocean coast are at Sandy Bay, Cape Ann, Mass.; Point Judith, R. I., and Delaware Bay, Del. A second breakwater is in progress of construction in Delaware Bay; while the report of a recent Board of Engineers decides upon San Pedro Bay, California, as the proper site for a breakwater to give protection to deep-water commerce, and furnish a harbor of refuge for the South California coast. The Delaware breakwater had its inception in a plan submitted in 1828 by a board of commissioners appointed by Congress. The project proposed the construction of two massive works, on the peirres perdue or rip-rap system, separated by an interval of 1,390 ft.; the larger work, called the breakwater, to afford safe anchorage during gales from the north and east; and the lesser, called the ice-breaker, to protect shipping against northeast gales and the heavy drifting ice of the bay. These works were commenced in 1828, continued under regular appropriations until 1840; resumed after the war in 1866, and completed in 1869. The aggregate amount expended was $2,192,103, which resulted in the construction of 2,558 ft. of breakwater, and 1,359 ft. of ice-breaker. In 1882 a project was adopted for closing the gap between the two detached works; and in 1883 work was commenced thereon. At the present time this work is in progress, and will probably be completed at an early date, whereby the entire construction will attain a length of sligthly more than one mile. The cost of the entire work will be about $3,000,000.

By combining a large number of sections of the Delaware breakwater, as determined by careful survey, the present slopes of the work may be classified as follows:

Plane of rest—sea face—at depth of 12 ft. below low water:

Sea face slope, below plane of rest...............1 on 1.6
Between plane of rest and low water.........1 on 2.9
Between low and high water.................1 on 3.17
Between low water and top of superstructure.1 on 1.77
Harbor face slope, between bottom and low water.1 on 1.35
Between low water and top of superstructure.1 on 0.73

It should be noted that the slopes directly above and below low water line on the sea face of the work vary but slightly from each other; the lower slope being 1 on 2.9 and the upper 1 on 3.17, or respectively about 19° and 17½°. This slight difference of about 1½° is probably within the error of determination, and without serious error the entire slope between the plane of rest and the base of the superstructure can be taken as about 1 on 3. The assumption that the plane of rest occurs at 12 ft. below low water is simply derived from the evidence of these

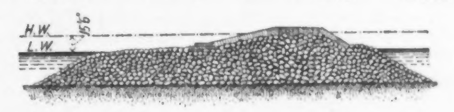


Fig. 6.—Section of Breakwater at Plymouth, England.

slopes. These slopes are graphically shown on the section below.

The mean cross-section of the first Delaware breakwater has the following dimensions:

Width at top of superstructure.............	22 ft.	0 ins
Height of superstructure above high water..	8 "	6 "
Difference between high and low water......	5 "	0 "
Width at high water	43 "	0 "
" " low "	62 "	9 "
" " plane of rest	113 "	9 "
" " bottom	166 "	0 "
Total mean height	43 "	6 "
Mean cross-section, in square ft............	4,100 "	

The ice-breaker, which from its position is much less exposed to the action of the waves, has a slope upon the sea face of 1 on 2, and on the harbor face of about 1 on 1.3; the mean cross-section of the ice-breaker is about 3,600 sq. ft. The calculated volume of the breakwater and ice-breaker, including voids, as determined from surveys, was 595,000 cu. yds. The quantity of stone delivered in the works was 892,528 gross tons. These figures would seem to indicate that 1.5 gross tons were required for each cubic yard of volume of the breakwater.

The foundation of the works must have sunk somewhat into the bed of the harbor, through the weight of the mass; recognized scour at the ends of the structure caused considerable material to fall into these holes; while other material has probably been carried by the sea outside of the limits of the work. These volumes have been omitted by the survey, and consequently have given too small a mass for the material deposited in the work. A survey of the material deposited in the gap between the breakwater and ice-breaker gave 1.17 gross tons per cubic yards of enrockment; while the careful measurement of a large volume of stone weighed onto barges gave 1.15 gross tons per cubic yard of volume. Observation derived from rock in settled railroad embankments give about 1.18 gross tons per cubic yard of space occupied. From these considerations Major Raymond derives the conclusion that 1.25 gross tons per cubic yard of random-stone embankment is an ample allowance for each cubic yard of breakwater after the settlement of the mass, and any loss of stone which may occur.* The stone which has been used in the Delaware breakwater weighs from 160 to 170 lbs. per cu. ft.

The Delaware breakwater has been the prototype for all other acomplished or proposed random-stone breakwater construction upon our ocean coast; and while its details have not been exactly repeated, the experience gained upon the work has been utilized both in design and construction at all other localities. The early records of this work seem to indicate that the Board of Commissioners, who designed the breakwater in 1828, were materially influenced by the dimensions and slopes used upon European breakwaters at that date, and that its foundation width was based upon anticipated slopes such as would he derived from the examples then before them; therefore, it

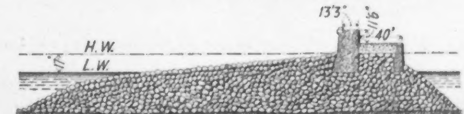


Fig. 7.—Section of Breakwater at Holyhead, Wales.

cannot be correctly assumed that the cross-section of the Delaware breakwater of to-day is exactly the result of slopes derived from the action of the wave forces which have been impressed upon them. The history of the work indicates that, at some time after its commencement, those in charge probably abandoned the idea of following the early adopted cross-section, and in place thereof sought to obtain such slopes as would result directly from the action of the waves; therefore, it follows that probably the resulting cross-section of the work of to-day is a compromise between anticipated and realized slopes upon the several parts of the structure.

The active sea slopes of the Portland, Plymouth and

*Report of Chief Engineers, 1891, page 1081.

Cherbourg breakwaters vary from 1 on 4 to about 1 on 8; while, as previously noted, the flattest slopes of the Delaware breakwater are about 1 on 3. In this respect it presents a striking contrast to European breakwaters in the steepness of its active sea slopes. While the Delaware breakwater is not exposed to the force of seas such as are thrown upon the European works named, it is highly probable that its slopes, through the method by which they have been obtained, are more nearly in equilibrium with the forces which are impressed upon them than obtains in the European cases referred to.

It is evident that the maximum economy in random-stone breakwater construction is realized when the slopes and consequent cross-section are such as to place the work in simple stable equilibrium with the forces which it is to resist. Any additional material beyond this would be unnecessary, and, therefore, an extravagance. To secure a stable section, and at the same time provide that it shall contain the minimum volume of material, requires that its method of construction should be carried on in the three following stages: (1) The formation of the volume below

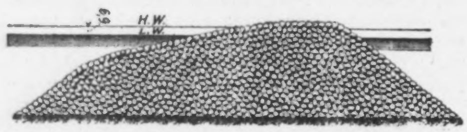


Fig. 8.—Section of Breakwater at Portland, England.

the plane of rest; (2) the deposition of the volume between the plane of rest and the base of the superstructure; (3) the construction of the superstructure. This order of construction has been developed and carried into practice by Major C. W. Raymond, Corps of Engineers, U. S. Army, who has present charge of the completion of the Delaware breakwater, and also the construction of the new breakwater in Delaware Bay. Major Raymond's extended experience in breakwater construction on the ocean coasts of the United States placed him upon the Board of Engineers for the location and plans for a harbor of refuge upon the coast of Southern California; and upon his plans and specifications the several reports upon this important work have been mainly based.

Under the order of construction above indicated the deposition of the random-stone is confined strictly to a width and depth conforming to the width and depth of the cross-section of the breakwater at the assumed plane of rest; by this method the material on the slopes is forced to assume the steeper angles of repose which properly adhere to this part of the work; at the same time the initial settlement of the foundation and this part of the mass is secured. When this lower section of the breakwater is completed it is followed by the second stage, the formation of the mass between the plane of rest and the base of the superstructure, by the deposition of all material within the width of the base of the superstructure; care being taken in this important section to deposit the larger stone upon the sea side of the breakwater. This brings the construction within the disturbing action of the waves; and as this part of the work progresses the stones upon the sea face of the work assume such slopes as to place the work in actual equilibrium with the wave forces impressed upon it; thereby leaving to the sea the task of largely declaring the slope and consequent stability which is required to meet the peculiar conditions of the exposure. If the engineer, for any reason, has erred in

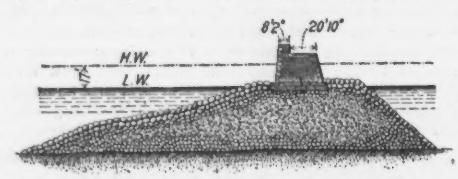


Fig. 9.—Section of Breakwater at Cherbourg, France.

judgment by assuming too steep slopes, the action of the sea revises his calculations, and, beyond the possibility of mistake, corrects the error. It is evident that by this method the minimum volume of section, and consequent maximum economy of cost, consistent with required stability is obtained.

By the time the entire breakwater has been completed to the height of the base of the superstructure an opportunity has been given this part of the work to thoroughly settle, so as to be in condition to receive the construction of the superstructure, which is not undertaken until a condition of stable equilibrium has been attained by the substructure, and its upper surface carefully levelled to receive the superimposed mass.

The superstructure, in the most recent types of American random-stone breakwaters, consists of rough, strong walls upon the sea and harbor faces of the work, built of larger stone systematically placed, so as to secure a strong bond; the space between these walls being filled with stone varying in size so as to form a compact mass without large interstices. This part of the work requires the use of powerful derricks for handling and properly placing the large stone constituting its mass. Under the methods above outlined, the several widths of deposits for the random-stone are marked by carefully placed ranges, and within their limits all material is strictly deposited. In this respect the practice now varies greatly from that of former times, when wide latitude was given to the areas upon which the stone was initially deposited in the efforts, then in vogue, to form predetermined slopes through the deposit of stone directly upon such slopes. Until later times it has been the practice to transport the stone to the site of the work upon sailing vessels, deck-scows, barges or other available means of water transportation; this necessarily resulted in more or less of a haphazard method of discharge and deposit, arising from the fact that the vessels used were not fitted with machinery and appliances for quickly and accurately placing the stone in the work. Such methods unquestionably resulted in considerable waste of material through its not being deposited exactly upon intended areas.

Under the methods now in operation upon the new breakwater for a National Harbor of Refuge in Delaware Bay, the material below the assumed plane of rest, as well as a part above this plane, is transported to the work in dump scows of 1,400 tons capacity, and specially arranged construction, whereby the entire load of such scows is released at once, and, without the intervention of labor or machinery, deposited, en masse, through the bottom of the scows directly upon the width of section on which the material is desired. Besides ensuring accuracy of deposit, the abolishment of the former large element of hand labor in the discharge of stone, permits the unloading of these scows during a rough condition of the sea, when, under past methods, it would have been either impracticable or, if attempted, resulting in an uncertain and unsatisfactory deposit of the material upon areas probably somewhat removed from those on which the stone should have been deposited. The remainder of the plant for the delivery of the stone in the new breakwater consists of large seaworthy barges, of 1,500 tons capacity each, provided with powerful steam derricks, capable of handling 15 tons, for quickly and certainly placing the stone at desired points.

The breakwater for the National Harbor of Refuge in Delaware Bay, which is now in progress, a section of which is shown here, is located outside of and nearly 2½ miles north from the existing breakwater. It is placed in water from 13 to 53 ft. deep, has a length of about 1½ miles, and covers a protected anchorage against the heaviest storms of 552 acres, with a minimum depth of 30 ft., and an additional area of 237 acres, with a minimum depth of 24 ft. These combined areas would give free and good anchorage to more than 1,000 vessels. The work is estimated to contain 1,384,000 net tons of stone, and will cost, with the projected ice piers, probably about $2,500,000. Although only commenced during the present season, the specifications for the work require its completion by December 31, 1901, through the annual deposit of about 280,000 tons of stone. Since the prevalence of severe or stormy weather practically reduces the working season available for depositing material in this breakwater to about seven months for each year, it follows that an average output of 40,000 tons per month must be provided for by the contractor. The work is being executed at this rate, which is far beyond anything ever before attained in earlier breakwater construction. The average price paid for stone in the old Delaware breakwater was about $2.50 per gross ton. Under the existing contract for the construction of the new breakwater, which covers the entire completion of the work, the price is $1.18⅜ per net ton.

The proposed breakwater for the projected deep water harbor at San Pedro, California, a section of which is also shown here, is located in water from 19 to 51 ft. deep. It has a projected length of 8,500 ft., and covers an anchorage area of 615 acres, with depths varying between 24 and 36 ft. The work is estimated to contain 1,782,000 cu. yds. of stone, or 2,290,000 gross tons. The material in the superstructure below the plane of rest is estimated at $1.25 per cubic yard; that above the plane of rest at $1.50 per cubic yard, and the stone in the superstructure at $3.00 per cubic yard. The entire estimated cost of the work, including contingencies, is $2,901,000.

The National Harbor of Refuge at Sandy Bay, Mass., was commenced in 1885, and is still in progress. It has a projected length of 9,000 ft., and covers an anchorage area of 1,377 acres, carrying a mean low water depth of 24 ft. and over. The material below the plane of rest, which in this case has been assumed at 15 ft. below water surface, consists of stones not exceeding 4 tons in weight; while the portion above low water requires stone not less than 4 tons and averaging 6 tons in weight. The contract prices paid for the stone already deposited in the work have been from 60 to 75 cents per net ton. The larger stones comprising the superstructure will greatly exceed the foregoing prices: the proximity to the work of large granite quarries has permitted the exceptionally low prices paid for the stone. The estimated cost of the entire project is $5,000,000.

The breakwater at Point Judith, R. I., was commenced in 1890, and is still in progress. It has a proposed length of about 10,000 ft., including an auxiliary breakwater protecting one of the entrances to the harbor. The protected area, carrying 18 ft. depth and over, is about one square mile. The work is built in water from 18 to 30 ft. deep, and its estimated cost is $1,250,000. In 1893 the government contracted for the entire completion of the work at the rate of $1.28 per net ton for all stone required.

By way of comparison the following sections of European breakwaters illustrate the difference between American and European practice in the construction of random-stone breakwaters, as well as the marked variation in the active slopes of such works. It should not be too strongly assumed that slope variations are due in all cases to corresponding differences in the forces impressed upon the active slopes, since such variations are partly due to preconceived ideas as to the necessities of the case, the dimensions of the stone used, and the methods by which the slopes were formed.

In addition to the constructions upon the American coast before referred, random-stone breakwaters exist at Gloucester and Hyannis harbor, Mass., and at Stonington and New Haven, Conn. These breakwaters are only typical of their localities since they are not designed to cover areas which would give protection to the larger class of ocean vessels, but rather to the coastwise commerce of their vicinities.

All breakwater construction in this country has been carried on under governmental appropriations made by Congress at intervals of one or two years. The separate appropriations were seldom more than five or ten per cent. of the estimated cost of the work, and until recently competitive bids were received and contracts entered into for the expenditure of each specific appropriation. Under this system each contractor was obliged to add to the cost of the work, and thereby to its cost to the government, the initial expenses of obtaining and opening quarries, and providing the special plant required, and with each change of contractors the government paid for the expense of organizing, fitting out and starting another. Under the limited tenure with which the contractor held the work, it was impracticable for him to adopt methods and appliances upon a scale large enough to permit the work being carried on with economy to the contractor and consequent advantage to the government.

In a construction of the magnitude of the breakwaters described involving the quarrying, handling, transportation and deposit of several million tons of stone, it is evident that the work can be most economically accomplished when the amount to be done under a specific contract is so large as to justify such an initial outlay for facilities and plant as will permit the work to be carried on upon a large scale. This has been rendered possible upon the Point Judith breakwater, and also upon the new breakwater in Delaware Bay, by Congress providing for a single contract for the entire work at each locality, under large annual appropriations, whereby the work can be completed in about five years. By this plan the contractor is justified in providing appliances and methods for carrying on the work upon a scale which could not be approximated under the former method of separate contracts for each specific appropriation. The resulting economy to the government is evidenced by the fact that under the former methods of contract, the average price of stone deposited in the old Delaware breakwater was about $2.50 per ton, and that delivered during the last thirteen years about $2.30 per ton; whereas the work now in progress upon the new breakwater, at nearly the same locality, costs the government under the method of continuous contract $1.18½ per ton, or a saving over former prices of nearly 50%.

Harbors of refuge are but seldom used by the larger class of ocean steamers, for, unless disabled by accident, they are generally competent to pursue their way during a storm, or else ride out a gale in the open sea with comparative safety; and, unless engaged in the coastwise trade, they would seldom be in a position to avail themselves of the protection of a harbor of refuge, even if it was desired. With sailing vessels or steamers carrying barges in tow and coastwise commerce such harbors are of the greatest utility, and they are placed at localities which make them most convenient of access to this class of commerce when approaching or leaving our ocean coast. During head winds or upon the announcement of dangerous approaching storms the sailing, barge and coastwise commerce have great need for harbors of refuge, and they are utilized to their full value.

THE CHICAGO AND MILWAUKEE ELECTRIC RY.

One of the longest and most interesting of the numerous long-distance electric railways in this country will be the line between Chicago & Milwaukee, two sections of which are now in operation. The Chicago & Milwaukee Electric Ry. Co. has the franchise for the line from Evanston to Waukegan, 31 miles, and the section of this between Waukegan and Highland Park was opened to traffic in July. Work on the southern part of the line is now being pushed, and it is expected that by October it will be completed to Evanston. At that point it will connect with one of the electric lines of the North Chicago Street Ry., with the Evanston branch of the Chicago, Milwaukee &

St. Paul Ry., and with the Northwestern Elevated Ry. when this latter line is completed. There is already a line in operation from Kenosha northward through Racine to Milwaukee, 33 miles, and the direct result of the opening of this line was to lead the Chicago & Northwestern Ry. to take off a large proportion of its local trains on account of the loss of traffic. Between Kenosha and Waukegan is a gap of 16 miles, but we are informed that negotiations are being made for closing this. The sketch maps given in Fig. 1 show the entire route, and also the route of the present line from Waukegan to Evanston. From the latter it will be seen that the line passes through a continuous series of the suburban towns which are such a feature of the lake shore north of Chicago. At Fort Sheridan is the government reservation, and there are manufacturing districts at North Chicago and Waukegan.

The project was originated by Mr. C. E. Loss, engineer and contractor, of Chicago, who made a beginning two years ago by building a line from Waukegan south to North Chicago, 4.4 miles, the average earnings on which line have paid 5% on $120,000. Later on, he built the line between Fort Sheridan and Highland Park, 4 miles, at a cost of $50,000, and this line has shown even greater net earnings. Mr. Loss then secured the balance of the franchises for the through line, and sold the entire project to a syndicate on the basis of the earnings, with a certain percentage for the securing of the franchises. He was then made a director in the new company, and given the contract for the entire line. The company was originally styled the Bluff City Electric Street Ry. Co., but this title was changed to the North Shore Interurban Ry. Co., and the present owner is the Chicago & Milwaukee Electric Ry. Co., the officers of which are as follows: President, George A. Ball; Vice-President and Treasurer, A. C. Frost; Secretary, George M. Seward; Directors (in addition to the above officers), Frank S. Reeves and C. E. Loss.

The line between Waukegan and Evanston will be 31 miles in length, and will lie between the Chicago & Northwestern Ry. and the lake shore all the way to North Chicago, where it will have two subway crossings, one under the Chicago & Northwestern Ry. and the other under the Elgin, Joliet & Eastern Ry. There will not be a single railway crossing at grade. The line will connect about 15 cities and towns having an aggregate population of about 75,000, and the fares will be about half the amounts charged for the same distances on the Chicago & Northwestern Ry.

The road is now a single-track line, with turnouts, but it is double track in Lake Forest, and next season will probably be double tracked throughout. While the greater part of its length will follow existing roads, there will be about eight miles on a private right of way of 25 to 50 ft. wide, and this strip of land will be protected by a five-wire fence of barbed wire. The company is doing the necessary grading to obtain as level a line as possible, and the maximum grade will be a short length of 3% at the subways. The minimum radius of curvature is 55 ft. The two subways will be similar to those in the track elevation work in Chicago, consisting of steel plate girder through spans carried on masonry abutments. That under the C. & N. W. Ry. will carry two tracks. There will only be 150 ft. of trestle work, which will be of substantial construction. Fig. 2 shows a section of a part of the line, the track being laid on a parkway at one side of a macadamized road. A block signal system is in use on that part of the line now in operation.

The track consists of 65 and 72-lb. rails of the "T" and "Shanghai" sections, and also 85-lb. girder rails, all in 60-ft. lengths, with splice bar joints having eight bolts in two rows. The ties are of oak, spaced 18 ins. c. to c., laid in broken stone and slag ballast. The "Protected" rail bond is used, which is a specialty of the Forest City Electric Co., of Cleveland, O. The 72-lb. rail and the rail bond are shown in Fig. 3.

In the business center of each town iron center poles are used, these being 28 ft. long, in sections 5, 6 and 7 ins. diameter. For the other parts of the line cedar side poles, with bracket arms, are used, the poles being 30 ft. long and 7 ins. diameter at the top. The high-voltage transmission wires are supported on glass insulators carried on wooden cross arms on the side poles. Each pole is painted, and those carrying the alternating-current lines have danger notices stenciled upon them. The span wires are of 5-16-in. galvanized wire, and the trolley wire is of No. 00 B. & S. gage.

The company is aiming to build a first-class railway, suitable for running trains at a speed of 25 to 30 miles per hour with comfort and safety. The line now under construction is being built for a contract price of $25,000 per mile, this being for a single track, but the price includes the private right of way, five acres of land near Fort Sheridan for the power house and car house, the double-track subway under the C. & N. W. Ry., and about two miles of macadam paving 18 to 20 ft. wide, with drainage, catchbasins, etc. It also includes a cash bonus of $10,000 as compensation on account of franchises.

Four-wheel and eight-wheel cars will be used, all having vestibuled ends and transverse seats, and the motor cars will haul trailers. The motor

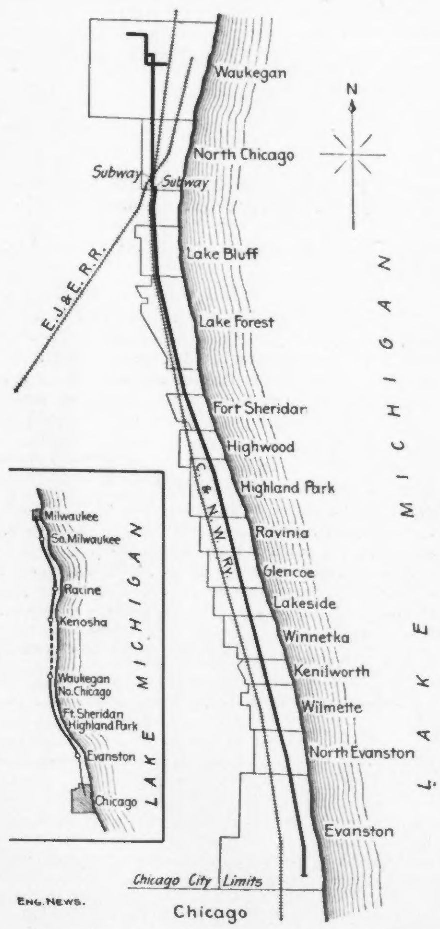


Fig. 1.—Sketch Maps Showing the Present Line and Completed System of the Chicago & Milwaukee Electric Ry.

cars will have General Electric motors of 35 HP. for the smaller and 50 HP. for the larger cars, of the latest type for high acceleration, enabling the car to attain its full speed of 30 miles per hour in 19 seconds. Arrangements have been made to carry freight and light baggage, and perhaps mails also. There are now in use ten 30-ft. cars built by the St. Louis Car Co., of St. Louis, Mo. Two passenger cars of the interurban type, and two combination passenger and baggage cars are being built by the Pullman Palace Car Co., of Chicago, Ill. These four cars will be 38 ft. long, each mounted on a pair of Peckham standard railway trucks and equipped with two 50-HP. motors.

The transmission system of this long-distance, high-speed, electric railway is of particular interest, the aim having been to devise a system which will require but a minimum outlay in first cost, but which can be added to as the load increases in a manner consistent with the original installation. It was also required that it should operate with a fair degree of economy from the start, and at the same time be capable of the highest operative efficiency when eventually completed. The great cost of copper for the feeders which would have been necessary to operate the road by means of a direct-current station situated at the center of the line prohibited the use of the direct-current system, and it was also recognized that the operation of two power stations for a road of this size, or of a booster system operated from one power house, would not give a good efficiency.

The plan adopted, and the one which is thought to combine all the above requirements, consists of a combination direct-current and three-phase alternating-current distribution system, with a single generating station placed at the center of the line. The sections of the road adjacent to the power station are supplied with direct current from the power house itself. Sections of the line at a distance from the power house get their supply of current from sub-stations. These substations each contain a rotary converter, consisting of a three-phase alternating-current motor and a direct-current generator combined in one machine. The sub-station motors are supplied with power over a high-tension transmission line consisting of three small copper wires extending back to the power house. The direct current from each sub-station is distributed to the trolley by means of the usual system of feeders extending in both directions. The location of these substations and their distance apart is governed by the conditions of traffic and somewhat by the nature of the grades along the line.

The power station is located at Highwood and the sub-station for the operation of the northern portion of the road is situated near the southern limits of North Chicago. Connecting the power house with the sub-station is an eight-mile, 5,500-volt, three-phase transmission line of three No. 8 wires. From the sub-station the No. 00 trolley wire is reinforced by a No. 0000 feeder extending four miles south and a No. 000 feeder extending 3½ miles north.

It is claimed that there is no reason why a road 100 miles in length should not be operated by this system from one power house located either at the center of gravity of the system or upon a site possessing peculiar advantages of rail and water facilities. The small investment required for copper will appeal to the investor, while the possibilities of economies of fuel and labor due to the concentration of the power generating apparatus into one station makes the proposition a promising one to the engineer. The development of this road and the economic results of its operation will be awaited with considerable interest.

An objection sometimes urged against the three-phase system is the possibility of danger from carrying such a high voltage through the feeder system, but it is to be remembered that this is a medium or low voltage compared with that of electric lighting and power systems now in use, notably about Niagara and in the West.

The power house and car barn will be of brick, with steel roof trusses, and a plan of the building and plant is given in Fig. 4. The present plan consists of one 400-HP. engine and one 250-K-W. dynamo, but the complete plant will comprise two additional tandem, compound, condensing Corliss engines of 800 HP. each, and four additional direct-connected General Electric composite generators of 250-K-W. capacity. Steam will be supplied by six Cahall-Babcock & Wilcox water-tube boilers of 250-HP. rated capacity, fitted with the Hawley down-draft furnace. These boilers will be arranged in three pairs or batteries, and three boilers are now in place. The boiler setting is such as to make each battery entirely independent of the brickwork which surrounds it. The feed water is taken from the city mains, and supplied to the boilers by the boiler feed apparatus of the Q. & C. Co., which has recently been described in our columns. The fuel is slack coal, which is delivered in front of the boilers directly from the cars. Each pair of boilers will have a fuel economizer, with the by-pass flue passing underneath. The steel smokestack is of sufficient capacity for eight boilers, in case it should be necessary to eventually add another pair of boilers.

The engines are of the horizontal type, and run at 125 revolutions per minute. The 400-HP. engine now in service has cylinders 17 x 36 ins. and 30 x 36 ins., and a flywheel 14 ft. in diameter, weighing 40,000 lbs. The governor is of the Proell inverted type, fitted with an automatic safety stop to prevent the engine running away. This stop blocks the governor balls up except in the event of the governor belt breaking, and the device requires no manipulation in starting or stopping the engine.

The present engine is of the side-crank type, but the two future 800-HP. engines will be of

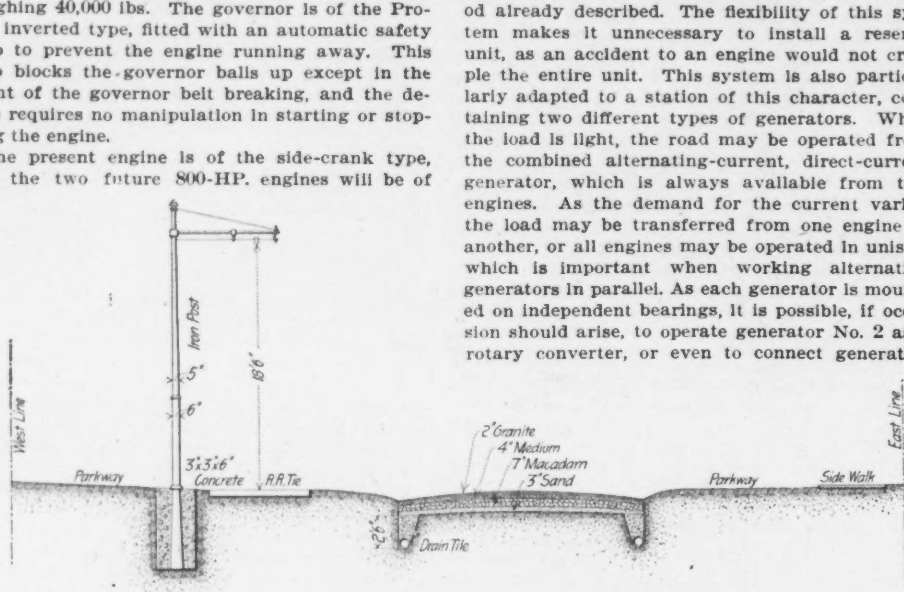


FIG. 2.—CROSS-SECTION OF ROADBED, SHOWING PARALLEL MACADAMIZED ROADWAY AND PARKWAY; CHICAGO & MILWAUKEE ELECTRIC RY.

the center-crank type, as shown in Fig. 4. Each engine is designed to sustain an overload of 100%, this provision being made for emergency service, when the engines can be made to develop a power greatly in excess of their rating by admitting high-pressure steam into the receiver through a reducing valve.

Special attention is called to the electrical generating plant, two types of generators being employed. Generator No. 1, which is next to the 400-HP. engine, marked No. 1 in Fig. 4, is an alternating three-phase machine, made by the General Electric Co., and has a capacity of 250 K-W. at 125 revolutions per minute. It has 24 poles, and, therefore, delivers a three-phase current of 25 cycles per second. This current is generated at a potential of 5,500 volts, ready to be delivered directly to the transmission lines without the use of transformers. Generators Nos. 2 and No. 4 are combination machines, and have also a capacity of 250 K-W. when running at 125 revolutions per minute. Each of these generators, however, can deliver its output either as direct current or as three-phase current. It has but one armature, but the windings of this armature are connected on one side to a comutator and on the other side to collector rings. The potential of the direct-current end is 600 volts, while the three-phase current is delivered at a voltage of 380. The generator can deliver its rated capacity of output from either the comutator or the collector rings. It can thus be used to supply current directly to the trolley passing in front of the power station, or can be used to operate the motors in the sub-stations, or for both purposes at the same time, dividing the load in proportion to the demand.

These two generators, as above noted, are of considerably lower voltage than the other three, and when their three-phase current is transmitted to the sub-station its potential is raised from 380 volts to 5,500 volts by means of static transformers placed in the basement, as shown in Fig. 4. The transformers are cooled by a blast fan operated by a 1-HP. induction motor.

The engines and generators are connected by means of the Arnold system, which has already been described and illustrated in our columns, and which has been adopted in a number of power stations. Each generator is mounted on a quill or hollow shaft, and suitable couplings connect this solid shaft with either of the engine shafts or the generator quills in such a manner that each generator can be operated by more than one engine. Thus, engine No. 1 can run either or both generators No. 1 and No. 2, while engine No. 2 can be used to drive four generators when they are eventually installed. In case of a break-down to any engine, the other engines in the plant can be made to carry an increase of load by the method already described. The flexibility of this system makes it unnecessary to install a reserve unit, as an accident to an engine would not cripple the entire unit. This system is also particularly adapted to a station of this character, containing two different types of generators. When the load is light, the road may be operated from the combined alternating-current, direct-current generator, which is always available from two engines. As the demand for the current varies, the load may be transferred from one engine to another, or all engines may be operated in unison, which is important when working alternating generators in parallel. As each generator is mounted on independent bearings, it is possible, if occasion should arise, to operate generator No. 2 as a rotary converter, or even to connect generators Nos. 1 and 2 so that they could be used in the same manner.

Each sub-station will contain the static step-down transformers, the rotary converter and a storage battery. It has not been considered necessary to have this equipment in duplicate, as it is intended to mount the machinery needed for a sub-station (static step-down transformers and one rotary converter) upon a truck fitted with motors. In case of break-down at any sub-station, the battery can be relied on to supply current for several hours during which time the portable reserve sub-station equipment can be propelled to the crippled station and connected ready to operate until the repairs have been made.

The storage battery in a sub-station of this character has many advantages. It supplies all the sudden and excessive demands for current, due to the fluctuating load, peculiar to a road of this kind, and operating heavy trains. The rotary converter can be made to carry a fairly average load, while in turn the generating machinery at the

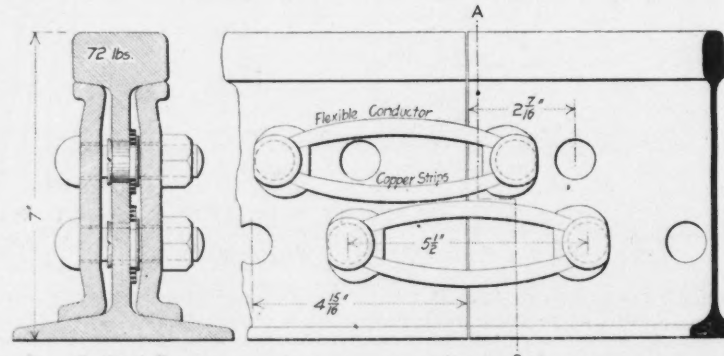


FIG. 3.—RAIL AND JOINT, SHOWING THE PROTECTED RAIL BOND; CHICAGO & MILWAUKEE ELECTRIC RY.

power house can be oprated under constant conditions, and this feature of steady load will tend towards fuel economy at the generating station. As the sub-station batteries are able to respond for short periods to an excessive demand, the generating station is relieved from the necessity of supplying the occasional maximum load, and consequently machinery of much less capacity is required at the station than if the batteries were not used. There is also a correspondingly reduced investment required for transmission lines and rotary converters.

There is still another advantage peculiar to this combined direct-current system of distribution. The self-induction of the alternating-current motors in the sub-stations causes "leading" and "lagging" currents, which may become extremely troublesome. The best condition for operating is when the power factor of the curcuit is unity; or when the apparent energy is equal to the real energy. The strength of the motor field can be adjusted at any given load so as to neutralize the inductance of the line, and leave the current nearly in phase with the E. M. F. If the motor can operate at or near this given load, the power factor will not depart far from unity and the circuit will act like a continuous-current line. If the load varies, however, to any great extent, the resulting "false current" may compel the transmission over the lines of a current greater than that really required to operate the motor. The storage battery furnishes a reservoir to receive or deliver energy as the load varies, enabling the rotary converter to carry a practically steady load, and thus helping materially in maintaining an electrical balance in the alternating-current circuits. It is considered that as the road is extended and the traffic becomes heavier, the conditions of the service will probably make the installation of a battery at the main power house a desirable investment.

The plans for the power plant and transmission system were prepared by Mr. B. J. Arnold, of Chicago, who is Consulting Engineer for the company, and it will be seen from the foregoing description that these plans include some decided novelties in electric railway engineering practice.

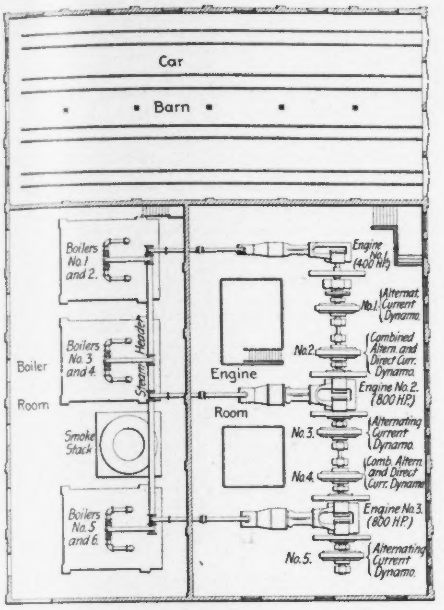
The principal contractors for materials are as follows: Dynamos, motors and electrical equipment, the General Electric Co., Schenectady, N. Y.; steam engines, the Filer & Stowell Co., Milwaukee, Wis.; boilers, the Aultman & Taylor Machinery Co., Mansfield, O.; rails, the Johnson Co., Lorain, O.; wire, John A. Roebling's Sons Co., Trenton, N. J.; cars, St. Louis Car Co., St. Louis, Mo., and the Pullman Palace Car Co., of Chicago, Ill.

The general contractors and builders of the line were C. E. Loss & Co., of Chicago. Mr. Loss, who was the original projector of the line, as already noted, has made a specialty of electric railway construction. In addition to the Chicago & Milwaukee line, he has also built the Pullman, the Calumet, the Englewood & Chicago,* and the Kankakee electric railways, in Illinois; the Hammond, Whiting & East Chicago, and the Broad Ripple railways in Indiana; the Sheboygan and Waukesha electric railway in Wisconsin; the Cleveland and Chagrin Falls and the Ironton electric railways in Ohio; the St. Charles and the Orleans railways in New Orleans; and the line from Jalapa to Coatepec in Mexico.

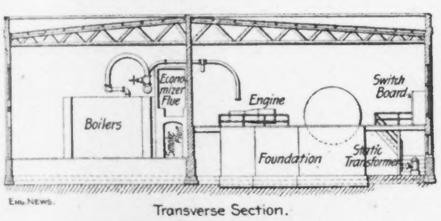
In the earlier part of this article we pointed out that while the present Chicago & Milwaukee line extends only from Chicago to Waukegan, there is also an electric railway in operation south from Milwaukee to Kenosha. This line is owned by the Milwaukee, Racine & Kenosha Electric Ry. Co., of which Mr. Matthew Slush is President. The line is 35 miles in length, of which seven miles are on a private right of way. The cars run into Milwaukee over the tracks of the Milwaukee Elec-

*This is a storage battery line, and has been described in full in our issues of Jan. 6 and Feb. 3, 1898.

tric Ry. Co., and have their terminal at the City Hall, the arrangement between the two companies entitling every passenger going into the city to a transfer to any point on the latter company's lines. The track is laid with 72-lb. rails and is ballasted with broken stone. The maximum grades are 3% and the sharpest curves of 60°. The power station is at Racine, and contains two compound engines of 1,500 HP. and two Westinghouse dynamos of 1,200-K-W. capacity. Long eight-wheel cars are used, having a 50-HP. motor on each truck. Each car will seat 50 passengers, and has toilet-room, smoking-room and a drinking-water tank. The cars are run singly, the time allowed



Sectional Plan.



Transverse Section.

Fig. 4.—Power Station and Car House of the Chicago & Milwaukee Electric Ry.

B. J. Arnold, M. Am. Inst. E. E., Consulting Engineer; C. E. Loss & Co., Chicago, Ill., Contractors.

being 2 hours 20 minutes, and there are 18 regular trips each way daily, besides which a number of special cars are run. Packages are carried, and in June a combination car was put on to carry mail and express matter, packages, fruit, milk, etc. The passenger fare is 70 cts., or $1 for the round trip.

SIXTH ANNUAL MEETING OF THE SOCIETY FOR THE PROMOTION OF ENGINEERING EDUCATION.

The sixth annual meeting of this society was held in the Walker Building of the Massachusetts Institute of Technology, in Boston, on Aug. 18, 19 and 20. This time was selected because the meeting of the American Association for Advancement of Science was to be held at the same place in the week beginning Aug. 22, and the meeting of the American Mathematical Society and the Conference of Astronomers and Physicists were to be held on the same dates (Aug. 18-20), thus making it convenient for the professors of engineering schools, who compose the greater portion of the membership of the Society for the Promotion of Engineering Education, to attend the meetings of the other societies. The society has now reached a membership of about 270, and about 70 members were registered as in attendance, a very large proportion for any scientific society whose membership is scattered over the whole country. The meetings were of exceptional interest and value. The papers presented were, many of them, of a high grade, and the discussions were brisk and full of interest. The members seemed to be animated with one common purpose, to improve the courses of education in our engineering schools, and in their consideration of this important subject they viewed it from all sides, not merely scrutinizing the minor details of the courses of study, but treating it from the broader standpoints of enhancing the dignity of the profession and of advancing the civilization of the world. Two notable addresses in which this broad view was taken were the address of the president, Prof. J. B. Johnson, on "A Higher Industrial and Commercial Education an Essential Feature of Our Future Material Prosperity" (Eng. News, Aug. 18), and Dr. R. H. Thurston's address on "The Correct Theory and Method of Organization and Conduct of Professional Schools."

The first session was held on Thursday morning, Aug. 18. After the reports of officers, election of members and other general business had been attended to, Prof. Johnson delivered his address. This was followed by a paper by Prof. Gaetano Lanza, of the Massachusetts Institute of Technology, of which we give an abstract below:

The Class-Room and the Laboratory in Their Mutual Adjustment to the End of the Most Efficient Instruction.

In giving a student an engineering education we wish to fit him, as thoroughly as possible, to grapple successfully with the duties and responsibilities of his profession, not only those of a technical nature, but also those which he must assume as an engineer and as a man.

Pure science and literature should not be neglected in his training.

To impart to the student a thorough mastery of scientific principles far outweighs in importance anything else that can be done for him, and this is the chief function of an engineering course.

The class-room work forms the basis of the course, and the laboratory work, to serve its purpose, must be based upon the class-room work which has preceded it, must be thoroughly co-ordinated with it, and must be made to depend upon it, to use it, and to serve as an aid to illustrate the principles involved. Without the laboratory work, the modern development of our engineering schools would have been impossible; for the laboratory work has not merely served to illustrate and elucidate the class-room work, but has also reacted upon it and caused the class-room work to be so modified as to accord with the facts; and thus has been produced an enormous development in the class-room work itself.

The functions of the engineering laboratory are partly to emphasize and illustrate the work of the class-room, partly to drill the students in performing carefully and accurately such experimental work as they are liable to be called upon to perform in the practice of their professions, and partly to teach them to carry on experimental investigations.

In order to fulfill these purposes there should be an intimate relation between the class-room and the laboratory work, and the student should be made to work up the results of the tests in the light of what he has learned in the class-room.

Any organization which does not tend to preserve the most intimate relation between the two, is not for the best interests of the students and should not exist.

A brief discussion followed, in which it was generally admitted that all of Prof. Lanza's points were well taken. The next paper, by Prof. H. B. Smith, of the Worcester Polytechnic Institute, was entitled:

Note on the Organization of an Electrical Engineering Laboratory.

The paper discusses briefly the importance of, and method of securing, system and flexibility in the equipment of an electrical engineering laboratory, whether for routine undergraduate work or for advanced research, and described the construction of a laboratory designed by the author to secure these ends.

In the afternoon visits were made by many of the members to several departments of the Massachusetts Institute of Technology. The evening was devoted to hearing a long address by Prof. R. H. Thurston, of Cornell University. The following abstract was furnished by the author and printed in the programme:

The Correct Theory and Method of Organization and Conduct of Professional Schools.

The correct theory and method of organization and conduct of professional schools, generally; the special methods applicable to (a) professional schools of engineering; (b) to trade schools; (c) to manual-training schools; (d) to mixed professional and educational curricula. The subtopics under this subject are:

(1) In the technical high-school, what properly determines the form and extent of its technical and other courses?

(2) In such schools, what should be the extent and character of what may be termed the distinctive and essential entrance requirements? What the non-essential requirements?

(3) In the technical college, what should be the nature and extent of these two different classes of demands? What courses should be offered in response to the demand for technical instruction from people whose sons must go directly into business from the lower schools if they cannot obtain a directly useful technical training?

(4) The professional engineering school, now being evolved—the distinctly professional school, having the same position and purpose as, in other professions, the law and medical schools—must soon, if not at once, require definition and delimitation from the academic colleges. What should be the entrance requirements?

(It is understood, of course, that what is practical is limited by the practicability of the existing schools providing it, and that what is desirable is determined by the demands of the profession relating to distinctively professional courses.)

(5) To what extent should the existing views and traditions of the people generally, and the prejudices of the academic teachers and monastic scholars, be deferred to, in the construction of the curriculum of the professional school of engineering, in which, as must be assumed, the work is to be professional training and not, in the accepted sense of that word, education?

(6) If, as is the fact in the state of New York, a body of academicians—its Board of Regents—assumes to control and to dictate to the faculties of the professional schools, what course should be taken to secure independence from such inexpert direction and to insure the management of professional institutions by professionally expert faculties?

(7) Assuming that it is obvious and is admitted that the form and extent of a professional course must be simply that which is demanded by the character of the work of that profession, and that it may be taken for granted that students entering such institutions will, as a rule, if not invariably, have sufficient intelligence and judgment to secure just as much of liberal education, of real education, in the accepted sense, as possible—what should be the length, the extent, and the limitations of such courses in the distinctively professional school?

(8) In such institutions, what other than professional work, if any, should be offered? What entrance requirements would suffice? Should the policy be that of a wide range of non-essential entrance requirements and high academic culture, with, necessarily, few students; or should the entrance requirements be restricted to characteristic and essential introductory work? With, presumably, unobstructed freedom of entrance of all classes and conditions of men; provided only that the poorest of them has attained competence in preparation in the essential requirements for entrance upon professional studies as taken up at the beginning?

(9) In the technical high-school, and in the technical college, of the more common sort in this country, in which some educational work is usually necessarily offered, where should the lines be drawn in respect to entrance requirements, and, in constructing the courses themselves, in such semi-professional schools?

(10) What are the similar conclusions respecting trade-schools and what respecting the manual-training schools now becoming common in our cities?

(11) How shall we classify all these various, as yet undefined courses and schools, in order to secure some definite principle of organization and operation?

The morning session on Friday began with a discussion of Prof. Thurston's paper. Prof. Woodward, of the Columbia School of Mines, pleaded for adherence to broad lines of general training and avoidance of too great specialization. He advocated that such inefficient management as the New York Board of Regents should be thrown out of existence. Prof. Fletcher, of Dartmouth, argued for the purely professional school of engineering, which leaves all elementary work to other schools.

The first paper presented was Prof. Haynes' description of the mounting and use of a spherical blackboard, read by Prof. Eddy, of Minneapolis. The importance of such a piece of apparatus in teaching spherical geometry, spherical trigonometry and astronomy was clearly shown, and the best methods of its use briefly explained.

A report of the Committee on Uniformity of Symbols was followed with interest by the audience, who seemed to have all suffered inconvenience from the confusion of various illogical symbols used in many sciences. The committee urged the universal adoption of symbols, as few in number as possible, but sees no way at present of securing this result. Two years ago a committee on this subject reported systems of symbols for different branches of engineering, which are published in the last volume of the "Transactions," but that committee's report was deemed unsatisfactory by many members, and a new committee on the subject was then constituted. This committee now reports that it is unable to prescribe a system of symbols which is likely to meet with general favor, and made two recommendations, first, that the consideration of the subject of symbols be dropped, and second, that a new committee be appointed to consider definitions of the terms used in engineering. The first recommendation was adopted unanimously, but the second was laid on the table by a large vote, and thus disapproved, and the committee was then discharged.

The first paper of the session was entitled "Some Phases of Engineering Education in the South," and was read by J. J. Wilmore, Professor of Mechanical Engineering, Alabama Polytechnic Institute.

The difficulties to be met by engineering education in the South are in some respects peculiar to the section, and in others, common to all sections. The problem of preliminary preparation is one, the solution of which all can share. In some of the southern states, however, it is more trying than elsewhere. This problem, complicated by the demand for a multiplicity of subjects other than engineering subjects, makes the arrangement of a satisfactory course of study difficult.

No college ever yet had as much money as it needed, but in the South the lack of funds is more keenly felt than in other sections, for the following reasons:

(1) The necessity of separate institutions for white and for colored students. The funds that might support one are insufficient for two.

(2) The absence of personal bequests and donations to white colleges in the South.

(3) The fact that lower fees are charged the students than in other sections of the country.

From the statistics of nine colleges in each section, selected to be as nearly representative as possible, the income per student for the northern colleges averages $242, and for the southern colleges $172. The average number of students per instructor in the northern colleges is 10.7, and in the southern colleges 12. The greatest income per student is $440, and the least $64. The smallest number of students per instructor is 6.2, and the greatest, 15.6. These statistics are based on the total attendance, and while not an absolute index of the engineering work, should give relatively the conditions in the two sections.

The inertia of the old prejudice of practice against theory is still an important factor in southern industrial management. This makes it difficult for our students to get positions which they are qualified to fill, without first undergoing a year or two of drudgery from which they secure little, if any, benefit.

The conditions, however, improve from year to year, and the prospect for the future is far from discouraging.

A paper was then presented by Prof. J. P. Jackson, of the Pensylvania State College, entitled;

"Electricity in Engineering Courses Other Than Electrical."

The present extended use of electricity in industrial undertakings has made a knowledge of its control indispensable to the well-rounded engineer.

College courses preparing for the older engineering professions were established while electricity was of small service. Through human inertia these courses have tended to continue on their old lines. The present time, therefore, finds training in electricity far from satisfactory in engineering courses other than electrical.

The work usually assigned is the electricity and magnetism of the general college physics course, with possibly one term on its applications. No time is given to a vigorous advanced treatment of the subject from an engineering standpoint. The college physics course might as well be depended upon to give the requisite training in applied mechanics or thermodynamics.

In addition to the work now usually scheduled, a thorough training in the laws of direct and alternating currents, as they apply to the practical utilization of electricity, should be added. This should be taught by an engineer, and should be assigned time comparable to that given the steam engine, hydraulics and hydraulic machinery, etc. This time could be obtained with advantage by cutting out special work. An outline of such a course is included in the paper.

As scientific knowledge increases, engineering courses must give more time to fundamental principles at the expense of mere technical practice. Hence such courses tend to converge.

If electrical engineering studies are to be satisfactory they must be under the supervision of an engineering department, and should not be taught by instructors in the departments of physics.

The next paper was by Mr. F. L. Dunlap, of the Worcester Polytechnic Institute, and was entitled "A Course in Industrial Chemistry for Technical Schools." The course, he said, should be directed toward practical results. His own students are sold raw products at their market value, and, after being purified in the laboratory, the refined material is purchased from them, thus stimulating their activity by a financial gain, through their chemical skill.

Prof. Brown Ayres, of Tulane University, Louisiana, in the discussion which followed, spoke of industrial chemistry in its relation to the development of sugar production. The American sugar chemists, he said, have nearly all come from abroad, because they are not rightly educated in the United States. The great increase in our sugar business, owing to the acquisition of new cane-raising territory, will greatly increase the demand for this class of chemist.

The next paper was by Prof. W. K. Hatt, of Purdue University, on

A Laboratory Apparatus for Impact Experiments.

Longitudinal impact is produced by the fall of an 845-lb. cast-iron weight or hammer on a specimen of steel rod. The rod is held at each end by conical wedges fitting in steel bushing pieces. The hammer is fixed to the lower end of the specimen, the lower bushing bearing against a shoulder made in the core of the hammer.

The combination of upper bushing, rod, and hammer, is lifted to a desired height and then released. The upper bushing is caught in the descent by cross pieces fixed to the uprights of the apparatus, and rupture of the specimen results. The circumstances of the impact are recorded on a revolving drum by a pencil attached to the hammer. The resulting curve furnishes data from which the ultimate resilience, elongation, time of impact, etc., are obtained.

The resilience is worked up in two ways: (1) by the application of the principle of work and energy to the descending weight, and (2) from the load-elongation diagram, whose abscissæ are the elongations of the specimen and whose ordinates are the corresponding forces, computed from the known mass of the weight and its retardation.

The apparatus illustrates to the student some principles in mechanics and some phenomena of rupture, and may be applied to purposes of research.

In the afternoon an excursion was taken by many members to examine the Boston subway under the Common, which was built to relieve the congestion of the streets by trolley cars, and the machinery for its ventilation, which is highly successful.

At the evening session the first paper was read by Prof. S. A. Reeve, of the Worcester Polytechnic Institute. It was entitled,

The Direct Study of Thermodynamics.

Prof. Reeve has been very successful in the use of the temperature-entropy diagram, the use of which he illustrated by several large drawings. The student should first, he said, be shown that heat and other forms of energy change places, and that energy is one and indivisible, appearing in different costumes as mechanical and electrical. He must then learn to see the two elements of force and mass in all energy, gaining a clear idea of what energy is.

There is nothing more mysterious in the passage of heat from a body of high temperature to one of lower temperature than in the flowing of water down hill or the corrosion of a metal by an acid. Heat runs down temperature as unavoidably as the stone falls from the hand when released. Heat engines and their opposite, refrigerating machines, are the apparatus to be used in thermodynamics instruction.

Prof. Peabody, of the Massachusetts Institute of Technology, and author of a text-book upon thermodynamics, proceeded to argue with a view to disprove among other things the assertion that it is of prime importance to develop an understanding of what energy and mass are in the minds of engineering students. Prof. Peabody fervently wished that he himself might first understand what these primary foundations are, but humbly confessed his ignorance. He stated that the chief reason that students found so much difficulty in thermodynamics was that they were not sufficiently well grounded in the calculus, and that thermodynamics lost all terrors to a student who had properly mastered his mathematics. Prof. Reeve was unfortunate in not having any defenders of his method of teaching among the audience. It would be well if teachers of thermodynamics would investigate it more carefully. If there is an easier way of penetrating thermodynamic mysteries than by first mastering differential equations it ought to be more generally known and followed.

The report of the standing committee on Entrance Requirements was then presented by Prof. Tyler. This committee is doing good work in collecting and publishing the statistics of the requirements of different schools, and in endeavoring to bring about some degree of uniformity. The subject was referred to the council, with instructions to take action upon several recommendations made by the committee, one of which is that of enlarging the committee so as to secure representation from different sections of the country.

The paper was read by Mr. W. B. Hoag, professor of civil engineering in the University of Minnesota, upon "A Course in Highway Engineering for Civil Engineers." The author made an earnest plea for a campaign of education upon the necessity of good roads, saying that road missionaries are needed before road engineers, and the farmers taught the immense profit that would accrue to them from good highways to their local markets. Now they are content to gage their loads by their estimate of the state of the roads, then trust to their luck and to Providence for the outcome.

Prof. Hoag briefly sketched the history of road-making from those used in the construction of the pyramids and the Roman successes in paving, down to the developments of modern macadamizing. He claimed that the art of road-making has not kept up with the advance in other arts, and that the United States is lagging in the rear of every other nation of equal general progress, in this very vital test of civilization.

The most interesting feature of the session on Saturday morning was a short extempore address by W. T. Sedgwick, Professor of Biology in the Massachusetts Institute of Technology, on "The Claims of Sanitary Science to a Place in the Curriculum of Engineering Education."

Prof. Sedgwick is a brilliant lecturer, and the abstract of his address printed in the programme, which we give below, gives a poor idea of the vigor with which he handled the subject. The abstract says:

The strength of the claim upon a curriculum already overcrowded depends simply upon the relative efficiency of the subject in contributing to the end in view, and this efficiency being in many cases difficult or even impossible to determine with any accuracy, opinions will differ, and curricula will differ, accordingly, in an entirely natural and probably very wholesome way.

A very great amount of modern engineering is essentially sanitary. Water supply, with its reservoirs, filters, dams, conduits, pipes, pumps, meters; sewerage, with its plumbing, drains, pumps, disposal works, sludge basins, and purification problems; warming and ventilation, with their questions of humidity and conductivity, affecting the health and ill-health of man, as well as questions of the purity of the air in respect to gases and micro-organisms; the construction of halls and buildings, of factories and tunnels, to be used by human beings, subject to infection, varying in lighting and air space; all of these have either their origin or their proper control in modern sanitary science.

It seems to the writer proper and wise, therefore, to provide a short course of instruction in the fundamentals of this subject; and such a course, consisting of fifteen lectures, with demonstrations, has now for some years been given by him with much success to civil engineers, architects, and others, in the Massachusetts Institute of Technology. A comprehensive survey of the course and its results appears in the complete paper.

In the discussion, a member said that the course of 15 lectures described by Prof. Sedgwick should be taken as a part of a liberal education by all students in colleges, rather than having it confined, as he suggested, to civil engineering students. The subject was not only of vast importance to the whole race of men, but, as presented by him, was essentially a culture study, in giving the student some insight into many different sciences and expanding his intellect in a greater degree than could be done by any course of 15 lectures on almost any other subject that could be named as a branch of liberal education. This view was generally favored by others who took part in the discussion.

Other papers presented at this session and discussed very briefly on account of lack of time are given in abstract below:

Undergraduate Thesis Work.

This paper was read by Edgar Marburg, Professor of Civil Engineering, University of Pennsylvania, and was in abstract as follows:

Former discussions of this subject indicate marked differences of opinion. These differences are probably more apparent than real. This paper is intended chiefly to elicit discussion.

The thesis is not regarded as a test of the acquirements or capabilities of the students. It is recognized rather for its value in affording more advanced and specialized training. The general character of the subject should be determined by the student's predilection, or by the nature of his prospective employment, or better, by both. Its choice should always be selected in consultation with the instructor.

In civil engineering, a design or an occasional resume requiring extended reading and study is preferable, generally speaking, to such so-called original experimental work as the average undergraduate is capable of. In a design, an excessive amount of routine work can be avoided; in an experimental subject, this is usually impracticable. The value of the former, educationally, is at least as great; as a means of professional training, usually much greater. In an experimental thesis there is more danger that the best interests of the student will not be held strictly in view.

The curriculum requires careful adjustment. Assuming a two-term year, courses most essential to thesis work should be completed during the first term. There should be less prescribed work for the second. The choice of subject and certain preliminaries should be disposed of during the first term; weekly periods should be assigned to thesis work throughout the second. The practice of not appointing fixed hours is deprecated.

The student should be left to his own resources within reasonable limits only. The work throughout should be under the conscientious supervision of the instructor. The undergraduate requires and should receive certain assistance that reference books do not and cannot supply.

The Construction of Models by Students as an Aid in Teaching Descriptive Geometry.

This paper was read by Henry S. Jacoby, Associate Professor of Bridge Engineering and Graphics, Cornell University.

Probably no course of study in the curriculum of engineering students makes such demands upon the imagination as a thorough course in the elements of descriptive geometry. The lack of preliminary training in this respect renders the subject difficult to a number of students. Some improvements, however, has been made recently in the attempt to educate the student's imagination to a proper comprehension of the forms in solid geometry.

There is a wide gap between the single projection used in geometry and the two or more projections employed in descriptive geometry. Models are used extensively, but the most of them relate to the latter part of the course, or to the application of the elements. The models available only in the class-room are insufficient for the best results. The student needs the model during lesson preparation. He can make the most useful ones with little expenditure of time and labor from cardboard, thread, and wire.

The chief value of a model lies in the effective aid in securing, at the beginning of the course, clear conceptions of the relation of the figure in space to its projections, and in that it can be viewed in those directions which show that the projections form adequate representations of the object for the purposes of the constructive arts. It is not necessary to have models for all the principal problems, but only for selected ones. In the study of the tangency and intersection of surfaces and the form of warped surfaces, the class-room models answer the purpose with but few exceptions.

The Training of Engineers for the Maintenance of Way Department on Railroads.

This paper was read by J. C. Nagle, Professor of Civil Engineering and Physics, Agricultural and Mechanical College of Texas, and was in abstract as follows:

This paper discussed the question of the proper qualifications of those entrusted with the responsible duties of maintenance of way on railroads, and took the ground that all such work should be entirely divorced from the operating department, being placed under engineering control. As to the training that engineers of maintenance of way should have, as well as that of others in responsible positions, it was maintained that an engineering college or university course is essential; but as these institutions are now equipped, the requisite knowledge of details cannot well be given. A plan whereby this may be accomplished was suggested, and an experiment upon one of the leading railroads of Southern United States was cited to show that young engineers can be found without difficulty, ready to serve as apprentices in any capacity, provided that hope of advancement is held out to them. It will be contended that roadmasters and superintendents should be engineers (and even section foremen should have such training as an engineer finds necessary), but at the same time they should have served in the lower ranks of the track force in order to thoroughly understand every detail of their work. This would require the co-operation of the railroads and technical schools, but in the end would tend towards the betterment of both—the engineering school adapting the courses of instruction to the needs of the railroads, and the latter profiting by the use of technically trained men in every branch of the maintenance of way department.

In the afternoon an excursion was taken down the harbor, by courtesy of the chief engineers of the Boston and Metropolitan Sewerage Systems, to visit the pumping plants and the outfalls of both systems, and also the plant now under construction for the utilization of garbage by the Arnold process.

THE POPULATION OF CONTINENTAL EUROPE is given by "La Revue Francaise de l'Etranger" as 380,000,000, made up as follows:

European Russia and Finland	106,200,000	Turkey in Europe	5,800,000
Germany	52,300,000	Roumania	5,600,000
Austro-Hungary	43,500,000	Portugal	5,000,000
The United Kingdom	39,800,000	Sweden	5,000,000
France	38,500,000	Holland	4,900,000
Italy	31,300,000	Bulgaria	3,300,000
Spain	18,000,000	Switzerland	3,000,000
Belgium	6,500,000	Greece	2,400,000
		Denmark	2,300,000
		Servia	2,300,000
		Norway	2,000,000

The density per sq. kilometer, which is about equal to 0.386 sq. miles, is: Belgium, 220; Italy, 169; Holland, 149; England, 126; Germany, 97; Switzerland, 73; France, 72; Austria, 69; Spain, 36; Russia, 20. While the annual increase of the population of Russia has been 1.45 for every 100 in the last ten years, that of Germany has been 1.15, of Austro-Hungary 0.96, of England 0.35, of Italy 0.45, of France 0.08. If this rate is continued, the populations at the end of 100 years will be: Russia, 228,000,000; Germany, 106,000,000; Austria, 79,000,000; England, 65,000,000; Italy, 44,000,000, and France, 40,000,000.

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Single track, miles	5,113	2,039	2,549	9,701
First-class passengers	26,526,020	4,459,260	1,512,393	32,497,673
Second-class passengers	58,789,395	....	3,973,255	62,762,650
Third-class passengers	812,745,556	101,994,671	20,419,651	935,159,878
Total passengers carried	898,060,971	106,453,931	25,905,299	1,030,420,201
Goods traffic, tons	315,869,515	53,466,221	5,046,530	374,382,266
Passenger receipts, gross	£34,461,205	£4,196,872	£1,454,525	....
Freight receipts, gross	£40,395,865	£5,846,802	£1,614,505	....
Total receipts of traffic	£79,759,776	£10,438,957	£3,538,321	£93,737,054
Total working expenses	£45,723,761	£5,384,639	£1,975,404	£53,083,804
Expenditures to receipts, per cent.	57	52	56	57
Locomotives	16,600	2,092	787	19,479
Passenger cars.	37,142	5,053	1,858	44,053
For passenger service	14,152	2,110	1,096	17,358
Freight cars	480,119	134,649	17,562	632,330
Miscellaneous	13,309	1,368	468	15,145
Total of vehicles	544,722	143,180	20,984	708,886
Capital, shares and stock	£708,423,602	£118,286,280	£30,579,111	£857,288,993
Loans and debenture stock	£282,444,279	£37,196,794	£13,085,109	£332,726,182
Totals	£990,867,881	£155,483,074	£43,664,220	£1,190,015,175

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THE NEW WARSHIP PROGRAM of the naval board of experts calls for the speedy construction of fifteen ships, of which six are to be heavily armored fighting ships; six are to be small protected cruisers, and three are to be midway between these extremes. The three battleships are to have 13,500 tons displacement on 25½-ft. draft, and have a minimum speed of 18½ knots with all stores and coal aboard. The three large armored cruisers will have 12,000 tons displacement, a speed of 22 knots and a steaming radius of 10,000 knots. There would also be three semi-armored and protected cruisers, of a highly improved "Olympia" type, and six 16-knot protected cruisers of an improved "Detroit" type, with 13,000 knots steaming radius. All these ships will be sheathed with oak and copper. The battleships will have armor over the entire ship, 12 ins. thick amidships, tapering to 5 ins. at the bow, and made of Krupp-Harvey steel. They will carry four 12-in. guns of extra length, for the use of smokeless powder; the two turrets will be of the elliptical balanced type, with sloping fronts. All the ships in this program will have water-tube boilers and twin quadruple engines. Bids for the three battleships authorized by the last Congress are now in the hands of the Navy Department. But these vessels, as called for, are mere duplicates of existing battleships of 11,500 tons displacement, 15 to 16-knot speed, and 16½-in. armor over vitals only. Under the present program the construction of these ships may be abandoned altogether, unless the bidders take advantage of the Secretary's appeal for greater speed. The $9,000,000 appropriated for these ships may thus apply to the present program, which will call for about $40,000,000 expenditure.

BIDS FOR RAPID-FIRE PROJECTILES, in lots of 100,000 each, were opened at Ordnance Bureau on Aug. 23. About thirteen bidders appeared, and the prices offered were as follows:

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A COLLISION AT SEA OCCURRED off the Banks of Newfoundland on Aug. 20 between the Thingvalla Line steamer "Norge" and the French fishing schooner "La Coquette," which resulted in the drowning of 16 of the schooner's crew. The collision occurred in a dense fog.

FLOATING AN IRON TANK, 82 ft. diameter, to a new foundation was lately successfully performed at Dunkerque, France. This tank was nearly 33 ft. high and had a total weight of 140 metric tons. Newly imposed fire regulations required the removal of this tank a distance of about 131 ft. to a new beton foundation previously prepared. To avoid the cost of dismantling and again setting up the tank the owner constructed a dike about the two foundations; the tank was hermetically sealed and air was pumped in by a compressor at the refinery. Water was then pumped into the diked area from the neighboring canal, and the air pressure was regulated as the water rose about the tank. When the tank floated it was hauled by cables to its new site and as the compressed air was released the tank settled into its place, without injury or deformation.

TELEPHONE STATISTICS OF EUROPE show an increasing demand for telephone apparatus. The following table, prepared by the U. S. Consul at Chemnitz, Germany, shows the present number of lines and instruments:

Country.	Number of lines.	Instruments.	Inhabitants to each telephone.
Sweden	293	42,354	115
Switzerland	295	23,446	129
Luxemburg	57	1,356	160
Germany	534	131,577	397
Holland	31	7,900	615
Belgium	15	9,400	682
France	407	31,681	1,216
Austria	124	18,950	1,318
Spain	48	10,810	1,597
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THE RAILWAYS IN INDIA, on March 31, 1898, aggregated 25,454 miles open and sanctioned, of which 21,156 miles were open for traffic, leaving 4,298 miles under construction or authorized. This was a net increase of 766 miles over the length of line in operation in March, 1897. The grand total includes 13,707 miles of Indian standard gage (5 ft. 6 ins.), 10,374 miles of meter gage, and 447 miles of special gages, mostly 2 and 2½ ft. Only about 1,300 miles were double-track. The rolling stock equipment includes 4,205 locomotives, 12,500 cars in passenger service and 78,789 cars in freight service, the total number of vehicles being 94,720. Of this equipment 1,035 engines and 5,480 cars are fitted with the automatic vacuum brake, while 630 cars have the train pipes but no brake apparatus. About 2,600 passenger cars are fitted with the Pintsch compressed gas lighting apparatus, and this system of lighting is being rapidly extended. The average percentage of operating expenses to gross earnings was 47.8% (29.47 to 69.59%) on the standard gage; and 50.76% on the meter gage (33.66 to 92.71%). The average number of passengers and the average journey per passenger, were 189 passengers and 42 miles on the standard gage, and 215 passengers and 36¾ miles on the meter gage lines. The average haul per ton of freight was 160 and 124 miles respectively. The employees were 296,495 in number, of whom 4,793 were Europeans, 6,902 were East Indians, and 284,800 were natives. The flood damages were exceptionally severe, and an earthquake also caused a considerable amount of damage. The accident list shows 79 passengers and 230 employees killed; and 338 and 368 injured.

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THE POWER PLANT OF THE HUDSON RIVER POWER CO., AT MECHANICSVILLE, N. Y.

The large water power and electric plant recently completed at Mechanicsville, N. Y., owes its existence to Mr. R. N. King, President of the Stillwell-Bierce & Smith-Vaile Co., of Dayton, O. Early in 1897 Mr. King became interested in the Mechanicsville water power, and after securing assurances of patronage from several large users of power, he organized the Hudson River Power Transmission Co.

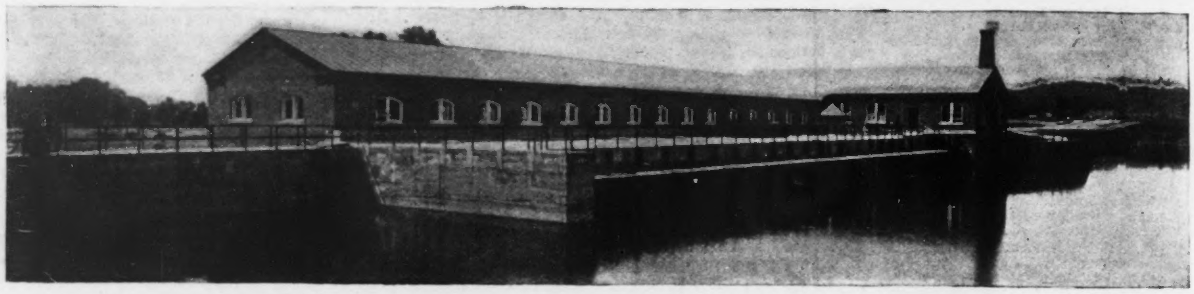

Fig. 4.—POWER HOUSE AND WESTERN DAM OF HUDSON RIVER POWER CO., MECHANICSVILLE, N. Y.; VIEW LOOKING DOWN STREAM.

The site selected is only 2 miles from Mechanicsville, 11 miles from Troy and 18 miles from Albany, in each of which towns there is a large market for power. Only 17 miles away is Schenectady, where the great works of the General Electric Co., occupying some 130 acres, were daily using tons of coal to operate steam engines, which in turn drove the dynamos necessary to supply current for operating the shop motors, testing apparatus, etc.

Naturally, as this company was to be one of the chief users of the power, their engineers were consulted regarding the electrical features of the installation. The hydraulic engineering features of the development were carried out from plans prepared by Mr. A. C. Rice, Chief Engineer of the Stillwell-Bierce & Smith-Vaile Co.

Location.—At the point chosen for the hydraulic development, the Hudson River is about 1,200 ft. wide, and is divided into two channels by Bluff Island, which stands about one-third the way across from the Western bank. The physical conditions make the location an ideal one for a dam and power house. During the greater part of the year the flow is sufficient to produce from 7,000 to 10,000 HP. In order to make the excavations for the dams and power house, coffer-dams were built across both channels. These were partly swept away by a heavy freshet, which caused considerable delay. It was also found that the rock excavated was unsuitable for concrete, of which the permanent dams were to be constructed, and it was necessary to build a special track and haul broken stone from a quarry near Schenectady.

The western channel of the river is closed by the power house, which extends from the shore out into the river about 250 ft., and by a concrete dam 26 ft. high above the bed of the river, 10 ft. wide on top, and 18 ft. wide at the base, extending from the end of the power house to Bluff Island. The upstream face of the dam is vertical and the downstream face has a batter. The top of this dam is at an elevation that the water will never reach, and it is not, therefore, designed to serve as an overfall, but the dam is provided with four arched waste gates 4 ft. wide and 6 ft. 9 ins. high, operated in the same manner as those in the main dam.

The main dam, Fig. 1, extends across the eastern channel of the river, and is also of concrete throughout. This dam is 16 ft. high above the river bed, 8 ft. thick immediately below the crest, and 16 ft. thick through the base. The dam is set between massive abutments anchored to the rock sides of the river bank and island. The upstream face is vertical, the downstream face is curved and provided with a horizontal apron 14 ft. wide, designed to give a horizontal direction to the fall, and prevent wash or scour at the toe of the dam.

This dam springs from two abutments, the eastern is 20 ft. long, 26 ft. high above river bottom, 16 ft. thick at the top, and 34 ft. wide at the base, while the western abutment is 100 ft. long, the other dimensions being similar to those of the eastern abutment. The length of the spillway between abutments, Fig. 2, is 800 ft. In the western abutment are 12 arched openings for waste gates. Each of these gates is 4 ft. wide and 6 ft. high, and is opened and closed by a heavy iron hoist operated by rack and pinion. The eastern dam is practically a solid rock wall capable of safely resisting any flood. It was severely tested in the spring of 1898, when an unusually large freshet came against it, yet, notwithstanding the fact that the dam was green, it was in no way injured.

To prevent any floating rubbish, ice, or logs reaching the racks or choking the waste gates, a floating wooden boom stretches from the western end of the spillway diagonally for a distance of 400 ft. to the edge of the normal river bank, and then for a distance of 1,000 ft. to the main embankment of the Mechanicsville highway. This boom is anchored to a line of stone-filled cribs.

From the west end of the power house to the Mechanicsville highway a broad embankment has been built to serve as a roadway, and cut off the flow along the river bank in high water. It was built of earth and slate rock with a concrete core, and is shown in section by Fig. 3.

Power House.—The power house, Fig. 4, lies between the west bank and the short concrete dam, nearly filling the space between the island and the west bank of the river, and is practically a continuation of the dam. It is of concrete, with the exception of the upper walls, and its construction is of the most substantial character. The foundations are carried down to the bedrock, and the house is carried on heavy steel box girders resting upon steel I-beam columns. The latter are imbedded in concrete walls carrying arches which form the floor of the generator room and the floor on which the wheel flumes rest. The walls form a separate and distinct tail-race 22 ft. wide for each set of turbines, from which the water may be shut out at will.

The house is divided into two parts by a thick head wall, Fig. 5. The upstream part contains wheel chambers for 7 1,000-HP. wheels, of which 5 only are at present occupied. The downstream portion contains wheel governors, and the electrical apparatus. The length of the power house proper is 257 ft. 6 ins.; the width of the dynamo room between head wall and south wall 34 ft., and the width of the wheel chamber portion 32 ft. 6 ins. The total width of the power house is 60 ft 6 ins. At the western end an L extension runs upstream 87 ft. 5 ins. long and 44 ft. 10 ins. wide. A retaining wall runs down stream from the power house along the western bank a distance of 50 ft.

Arched chambers are provided for 7 main wheels and two exciter wheels. Each main chamber is 32 ft. 6 ins. long, 22 ft. wide and 17 ft. 5 ins. high, and is provided with two 6-ft. manholes. Each exciter wheel chamber is 32 ft. 6 ins. long, 17 ft. 5 ins. high and 10 ft. wide. The head wall of the chambers is 6 ft. thick, the wall on the upstream and both sides 3 ft. thick. In the head wall of each main chamber is set a heavy cast-iron cover through which the turbine shaft passes in a water-tight packing box, carrying the ring oil bearing for the shaft. The saving in space effected by the new arrangement of housing and coupling together of the turbines, patented by the Stillwell-Bierce & Smith-Vaile Co., has diminished considerably the size of the power house. A 20-ton crane from the works of Pawling & Harnishfeger, Milwaukee, runs the entire length of the dynamo room.

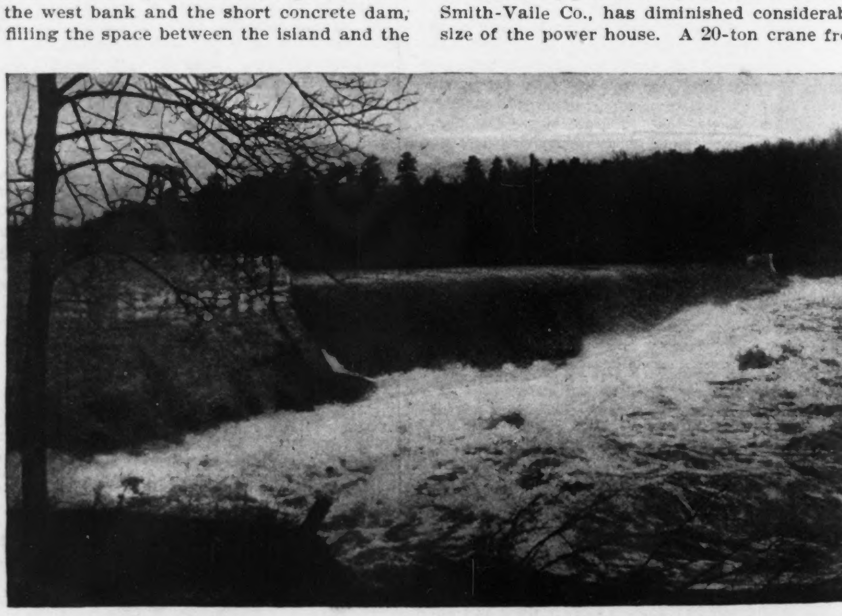

Fig. 2.—DOWN STREAM SIDE OF MAIN DAM SHOWING WASTE WATER PASSING OVER.

In front of the wheel chambers and running the entire length of the power house, is a trash rack of steel bars supported on a framework of steel channel and I-beams. This rack effectually prevents the access to the wheels of any rubbish or floating material that may escape the boom.

Water Main Wheels.—There are ten pairs of 42-in. horizontal Victor turbines, built by the Stillwell-Bierce & Smith-Vaile Co., of Dayton, O. Each main turbine consists of two pairs of wheels, which, at the normal speed of 114 revolutions, and the ob-