REINFORCED CONCRETE BUILDINGS

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(*Frontispiece*)

ARCTIC OIL WORKS, SAN FRANCISCO
Reinforced Concrete Walls and Chimney erected by Ernest L. Ransome, 1884

REINFORCED CONCRETE BUILDINGS

A TREATISE ON THE HISTORY, PATENTS DESIGN AND ERECTION OF THE PRINCIPAL PARTS ENTERING INTO A MODERN REINFORCED CONCRETE BUILDING

BY

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PREFACE

THIS little volume is presented to the engineering profession for the purpose of showing what reinforced concrete is, and how it came to be what it is. In deciding upon the scope of the book, the authors have endeavored to select matters of interest to the mature and experienced engineer, and for that reason the ultimate *result* of the analysis has been treated in greater detail than the derivation itself.

Many books on reinforced concrete have been written principally for the practical man or even for the untrained man; those who approve of that tendency will not approve of this book, in which all references to weights and dimensions, earth pressure, etc., have been avoided. The practicing engineer, the contractor, or even the college student should look for such matters in special "pocket books," and he must not expect the reinforced concrete book to furnish a complete encyclopedia on civil and hydraulic engineering.

On the other hand, much matter has been included in this book which will be looked for in vain in other works, and this is especially true of Part I, where an account of the history of reinforced concrete has been given, with special reference to the patents granted by the United States. Naturally, a selection of the more important or interesting patents is a difficult matter, and most likely some readers will find that too much has been included, and others that not enough has been included. However this may be, the records of the patent office contain on the whole more and better information than any other source, and the engineer who is striving to attain perfection cannot afford to relegate the most valuable thoughts and results of his predecessors to the scrap heap. It is also hoped that inventors and patent attorneys may find matters of interest in the necessarily brief descriptions given; anybody wishing full information in regard to any patents may obtain copies for a nominal sum by addressing the Commissioner of Patents, Washington, D.C.

The theoretical analysis in Part II has been made as brief and concise as seemed consistent with its purpose; — the solutions of equations, etc., have usually been given after the premises have been stated, omitting all the intermediate steps. If this book should find its way into the classroom, the teacher can easily supply what is omitted; — the practical engineer would not stop to read a treatise on mathematics, in any case. The author believes that much of this is new and original, especially the use of two constants in the bending problem; the further development in Articles 34, 35, and 49, where the effect of an increase in depth of beam is discussed, and the analysis of stresses in given beams.

The entire chapter on Transverse Stresses and U-bars is original, and avoids the use (or mis-use) of the word "shear." In reinforced concrete the steel is supposed to act in tension, and the U-bars must follow this general rule.

Part III is devoted to the practical construction. Here again more attention has been given to the useful facts not generally known, than to those that are matters of common knowledge. There is no necessity today for describing at great length the various types of buildings, or their component parts; an exception has however been made in regard to "Unit Construction" which appears to be coming rapidly to the front. No effort has been made toward giving the details of form design, but the general principles have been stated with great care. The chapters on fireproofing and repairs should be of interest, and the superintendents' specifications have proved their own value on a number of large contracts. Immediately preceding this chapter we have placed a short account of some bad failures; while we have not been able to throw new light on the causes, we hope that the perusal of the chapter on "accidents" may put the reader in the proper frame of mind to not only read, but also follow, the instructions given in the last chapter.

It has been customary with other writers to describe in more or less detail the tests made on reinforced concrete beams. As a general principle we have avoided such discussions, partly because the plan of this book did not allow us to devote the required large number of pages, and partly because the vast majority of tests are of little value, not from want of ability or care in the experimenters, but because the tests were not systematized, that is, every group should first demonstrate one general fact, and then individual test specimens should be so designed that they vary in only one feature from the standard, so that the effect of the variation at once becomes evident. The groups of tests so made are few indeed, and only during the last few years have clear photographs of the broken specimens been published. However, where a given problem requires the illustration of a test, the best available source has been referred to.

In this book an earnest effort has been made toward stating the truth when it was known, and to make it clear and evident that the truth is not known in a number of cases. The chapters relating to the mathematical design are so arranged that everyone can readily assure himself of the correctness, but in regard to such matters as cement testing, rolling of concrete floors while setting, and numerous other practical or general propositions where the authors have taken issue with prevailing ideas, and gone contrary to accepted practice, our statements must either be rejected as heresy or accepted as doctrine.

The authors desire to acknowledge their indebtedness to various papers published in the Engineering Record, the Engineering News, the American Machinist, and the Cement Age. Information has also been gained from a paper read by Geo. W. Percy before the San Francisco Chapter of the A.I.A. (Feb. 9, 1894); from C. W. Pasley's "Observations on Limes" (1847), from Hyatt's "Account of some Experiments" (1877); from papers by Scott, Bernays, and Grant, edited by James Forest as a separate volume under the name "Portland Cement" (1880); from "Reinforced Concrete in Factory Construction" by the Atlas Portland Cement Co., and in regard to theoretical questions, from works by Considère and Mörsch. The author of Part II desires to emphasize the inspiration received from the study of these two authorities, who have contributed so much to the knowledge of the subject. The discovery of the "water-marks" by Professor Turneaure has perhaps influenced the U-bar theory here advanced more than any other tests on record have.

The authors are greatly indebted to Professor L. J. Johnson, M. Am. Soc. C. E., for certain data relating to reinforced concrete beams tested at Harvard University. The results obtained are extremely important and will undoubtedly revolutionize

current practice in regard to the simultaneous manufacture of beam and slab. Moreover, these tests confirm in a remarkable manner the theories advanced in Chapter VII, which were conceived and printed long before the test beams were designed.

E. L. R.
A. S.

March, 1912.

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PART I

A CONTRIBUTION TO THE HISTORY OF REINFORCED CONCRETE

REINFORCED CONCRETE BUILDINGS

CHAPTER I

PERSONAL REMINISCENCE

BY ERNEST L. RANSOME

When, in 1859, I entered as an apprentice in my father's factory in Ipswich, England, the concrete industry was in its infancy, and was confined largely to the manufacture of artificial stone for ornamental purpose. One of the earliest applications of the new industry was invented in 1844 by my father Frederick Ransome, who was then engaged as superintendent of the well-known Iron Works of Ransomes and Sims at Ipswich. Noticing one day the waste of good hard stone in the dressing of mill-stones, he conceived the idea of cementing hard, selected pieces together, and so to manufacture a superior grade of burr-stones. The first difficulty was in finding a proper cementing substance: plaster of paris, shellac, glue, isinglass, lime with bullock's blood, mastic, etc., were tried and discarded. Among the numberless ingredients tried were also common glass, but it was not until experiments with soluble glass were made that success became probable. It occurred to him that if he took flint stones with a moderate amount of caustic alkali in solution, and subjected them to heat in a Papin's digester under high pressure, he might be able to concoct a soup from flint, as Papin had done from bones. But the result was apparently a disappointment, and in order to increase the heat, he finally tied the safety valve with a piece of wire, and forced the fire until the boiler became overheated. Fearing, however, that the boiler would blow up, he threw it out into a cistern with cold water, and the boiler, as might have been anticipated, was broken to pieces — and there, inside, was the glazy, syrupy mass of dissolved glass. The portions next to the walls of the boiler were baked to a flinty hard stone; in one word, the problem was solved.

Step by step, a process was now evolved whereby a cementing substance was had, as above described; and based upon this process, a large business was developed. Before long, the parent Company, "Patent Concrete Stone Co.," was selling its product in all parts of the world, especially after methods had been invented whereby the stones were made not only hard but also weather-proof. This process consisted originally in the application of a solution of chloride of calcium to the silicate of soda previously used, whereby insoluble silicate of lime, and soluble chloride of sodium were formed by double decomposition. The latter is common cooking salt and was easily removed by washing.

A further experiment disclosed the fact that powdered magnesian limestone, mixed with a small quantity of silicate of soda, formed a very hard substance when submerged in a solution of chloride of calcium, in a very short time.

In America, the new process was introduced in 1870 by the Pacific Stone Company of San Francisco, of which Company I was the superintendent for four years. About this time, the concrete industry was in slow development on the Coast, based upon the use of imported Portland Cement; in 1874 I remember to have paid as much as nine dollars per barrel of cement. But even as late as 1882, the concrete construction was mainly utilized in foundations and arches suspended between iron beams. In the latter type of construction some trouble was experienced with the cracking of the concrete over the beams, and to overcome this tendency I patented, No. 263,579 (Figure 1), a con-

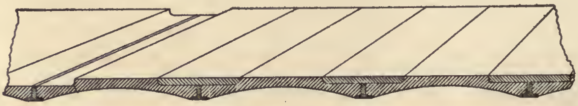


FIGURE 1.

struction in which an expansion-joint feature was introduced, and several sidewalks have been built over cellar areas in this manner.

Before long, I was called upon to devise a cheaper method of self-supporting sidewalks for the Masonic Hall at Stockton, Cal., and this I accomplished by using, instead of the I beams, a 2″ round tie-bolt to carry the tension, while the concrete carried the compression. The rods had upset ends and large cast-iron washers at each end, and I soon found that the upsetting and threading of the ends, the nuts and washers, etc., made the cost of the finished rod exactly twice that of the plain rod. I looked around for means whereby a continuous tie or bond could be developed along the length of the rod, and even contemplated cutting a spiral groove in the rod, when suddenly the idea of twisting a square or rectangular bar entered my head. I happened to have a rubber band in my pocket, and the spiral thread became at once evident when the rubber band was twisted in the hand. My patent, No. 305,226 (Figure 2), was granted in 1884, and the mills were soon turning out twisted bars up to one inch square, at a cost of about ten dollars per ton for twisting. Larger bars they positively refused to tackle under the plea that the common lathes used for the purpose did not have the requisite strength. I had, however, in my yard an old concrete mixer equipped with a worm and wheel, and by modifying this arrangement I soon succeeded in twisting 2″ square rods, using hand power. The cost did not exceed seventy-five cents per ton, and from that date until a more recent period, all the twisting was done in my own yards.

FIGURE 2.

However, the introduction of the twisted iron was no easy matter, and when I presented my new invention to the technical society in California, I was simply laughed down, the concensus of opinion being that I injured the iron. One gentleman kindly suggested that if I did not twist my iron so much I might not injure it seriously, in spite of all my references to the twisting of ropes and similar devices. This argument I based upon the supposed fibrous or laminated structure of the iron.

But all this criticism led to exhaustive tests, and when the professors found that my samples stood up better than the plain bars, one even went as far as to suggest that I had doctored my samples. This led me to twist half of each test rod only, and the superior strength of the cold twisted iron was finally admitted, and in due time, when steel became common, even better results were had with cold twisted steel. Even at this present time, I do not believe that the increase in strength due to the twisting has been accounted for. In this connection I call attention to an interesting fact first discovered by Professor Hesse, and that is, that bars tested at once after twisting do not give as good results as those tested five or more days after twisting, showing that a certain slow change takes place in the structure of the iron.

From the earliest time of my career I have experimented extensively with concrete mixers and other machinery, and the Ransome mixer is now a standard article. However, a description of these experiments would carry us too far, and might not interest the reader. Suffice it to say that my first patent for a concrete mixer was granted in 1884, to be followed by many more.

Up to about 1888 my work in reinforced concrete was largely confined to what we now term small and unimportant structures. The Bourn & Wise wine-cellar at St. Helena, Cal., was erected in 1888; the building is 75′ × 400′, three stories high, with stone walls. The main floor only was of reinforced concrete resting upon iron columns. The design is shown in Figure 3. The next floors were erected for the Californian

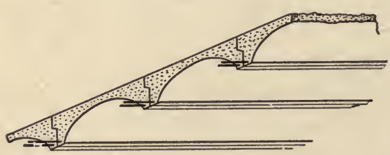


FIGURE 3.

Academy of Sciences in San Francisco, Figure 4; during construction these floors were subject to much adverse criticism from many architects, builders, and members of the Society, and efforts were made to have the fire wardens condemn the work. However, inspectors from the fire department found nothing about the construction that could be injured by fire, and having sense enough to perceive its great strength, they declined to take any action in the matter. To satisfy all skeptics in regard to the strength, a section of the second floor 15′ × 22′ was uniformly loaded with gravel to 415 lbs. per square foot; the deflection was $\frac{1}{8}''$. For the further satisfaction of the doubtful the load was left on for four weeks, but very few availed themselves of invitations to examine the work a second time. The few who came were, however, convinced.

The Leland Stanford Jr. Museum, at Palo Alto, Cal., was

FIGURE 4. — ACADEMY OF SCIENCES, SAN FRANCISCO
Reinforced Concrete Floors, Cast Iron Columns
Erected by Ernest L. Ransome

erected about this same time, and the entire wall and floor construction was of concrete, the walls having superficial joint lines as indicated in my patent, No. 405, 800, of 1889 (Figure 144A). The outside surface was partly tooled, and the whole was built in the classical design originally made for sand stone. The greatest innovation was, however, the roof, and this was probably the first instance on record where a finished and exposed roof was made entirely of concrete. The roof was supported on iron trusses 10 ft. on centers and the concrete construction rested upon the iron rafters, as shown in Figure 5. The roof over the central pavilion is quite flat and is $46' \times 56'$ in plan, reinforced with 2″ twisted bars 60 ft. long and with a flat reinforced concrete dome panelled with 1″ thick glass. In the same location the Girls Dormitory of the Stanford University was erected soon

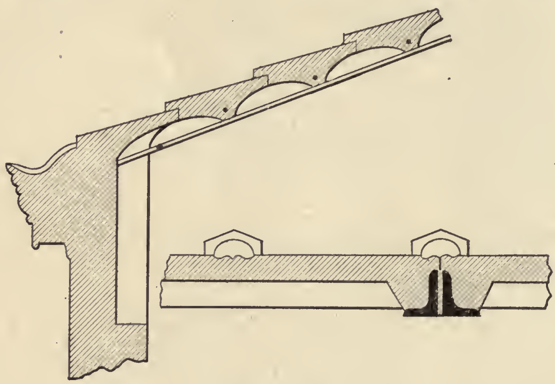


FIGURE 5.

afterwards, and its three stories were completed in ninety days from the time the plans were ordered.

When, on April 18, 1906, San Francisco was destroyed by the earthquake, the buildings at Palo Alto suffered severe damage, in many cases beyond repairs. However, the old reinforced concrete buildings referred to above stood the test with little if any damage; see Bulletin No. 324 of the United States Geological Survey, pages 22, 23, 24, 75, 112–114.

It may be worth while to note that the addition to the Borax Works at Alameda, Cal. (1889), was the first instance of the ribbed floor construction erected; it will be seen from Figure 6 that the construction is identically the same as used for that kind of floors today. The columns were also of concrete, probably the first ever erected.

I desire to express here my sincere gratitude to the men who, in those early times, had the confidence and foresight to realize the technical and commercial importance of the novel construction, often in the face of severe criticism and bitter attacks. Chief amongst these are my associate for many years, Mr. Frank M. Smith, Architect Percy and Governor Stanford, deceased.

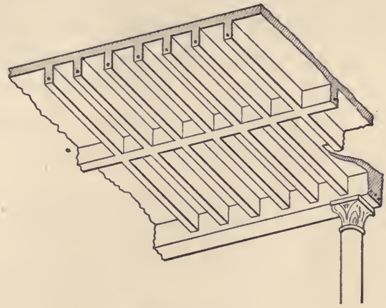


FIGURE 6.

Additional information in regard to the preceding buildings may be found in a paper "Concrete Construction," read by George W. Percy before the San Francisco Chapter of the American Institute of Architects, February 9, 1894.

In this paper reference is also made to my tests on delaying the placing of 1:2 mortar; the results are really astonishing. The tensile strength of briquettes made with Knight, Bevan, and Sturgess cement was as follows:

Delay in hours	0	1	2	$2\frac{1}{2}$	$3\frac{1}{2}$	$4\frac{1}{2}$
Tens. Strength, lbs.	252	228	240	256	306	228

showing that a delay of $2\frac{1}{2}$ to $3\frac{1}{2}$ hours really gave the highest results. Similar tests with White's cement gave the best results with a delay of $1\frac{1}{2}$ hours, after which the strength fell rapidly. In all these cases, the concrete was worked up again as soon as it stiffened. Unfortunately, these tests have never been extended to modern cement. However, in my address to the Society of American Architects, October 17, 1894, I called attention to the truly remarkable results obtained by Mr. Spencer Newberry, who found that a mixture 1 : 3 which, when worked for one minute with a trowel, developed a tensile strength of 87 lbs. in seven days, developed a strength of 240 lbs. in the same period after being worked with a trowel for five minutes.

I also made experiments with continued mixing, keeping the concrete in the mill for as many as 1000 revolutions. I found that within this limit the strength of the concrete increased with the number of revolutions, so that concrete given 1000 turns in the mill was stronger than when it had had 700 revolutions only.[1]

These observations led me to believe in the continued working of the concrete while it is setting; that is, when making a slab, I put men on with rollers who were instructed to keep the rollers going for several hours. The slab is laid on the forms in the usual manner; as soon as the concrete is hard enough to carry a man, the rolling begins, and is carried on with two or

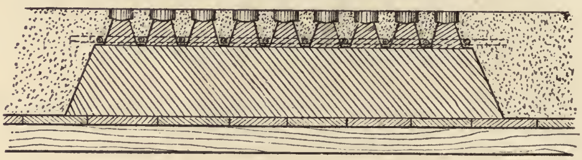


FIGURE 7.

three sets of rollers of increasing weight until the rollers make no impression. A more handy method is to use hollow iron drums filled with increasing amounts of water. It is important that the floors be not allowed to dry out too quickly, for which

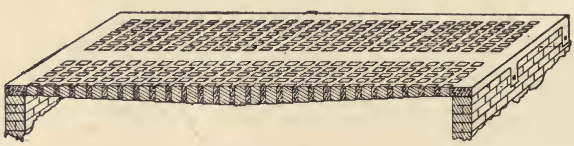


FIGURE 8.

reason they may be sprinkled if necessary during rolling and kept moderately wet for at least one week more.

The construction of "illuminating panels" in concrete floors, or, as they are more commonly called, sidewalk lights, has captured the attention of inventors for many years. But owing to the unequal expansion of glass and iron, the great majority of such constructions embodying a combination of these two elements have not been satisfactory. My patents, No. 448,993 (1891), Figure 7, and 518,045 (1894), Figure 8, aimed to avoid

[1] It must here be noted that the mill used for this experiment was of a different type from those used today, the modern machines having a much more severe action. The danger in overmixing is that the aggregate is ground very fine, thus giving a mortar with an excess of sand.

the use of the iron plate by setting the glasses in a body of reinforced concrete, and this was accomplished with great success. The Ransome Sidewalk Lights may be seen in every large city of the country, amongst other places the New York Subway, and my patents formed during their terms the basis for a large and prosperous industry.

One of the problems most troublesome to the reinforced concrete engineer is encountered in joining new concrete to old. A more or less suitable joint may be had in a number of ways, and from time to time I have given this problem much thought. A purely mechanical bond is created by bedding an open coil half way in the old concrete surface, so that the other half is caught in the new concrete, subsequently molded against the old surface, Patent No. 647,904 (Figure 9). This principle is

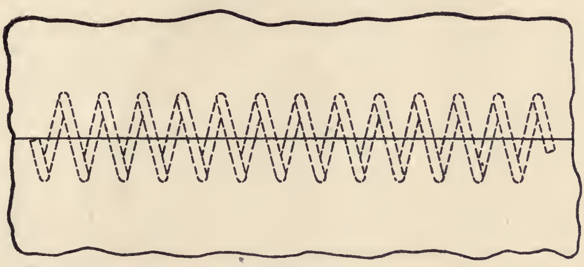


FIGURE 9.

utilized in the "unit" construction of reinforced concrete buildings according to my patent No. 694,577 (1902), Figure 10, and the tie thus made is so efficient that the subsequently

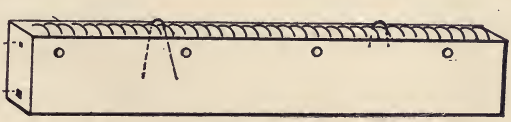


FIGURE 10.

molded slab may serve as the compression flange for the beam or girder erected ahead of the slab. The first application of this principle was in connection with the office building erected for the Foster-Armstrong Company at East Rochester, N. Y. (1904–5), and it has since been extensively used. Honeycomb slag may also be utilized in a similar manner, Patent No. 694,578 (1902), especially for large surfaces, but here I prefer my later invention, the removal of the surface skin with hydrochloric acid. The surface is next washed with water, and the finish coat is then placed in the usual manner, Patent No. 800,942 (1905).

This latter method, "the acid joint," has been tried in practice with excellent results. Fresh bases were welded to old concrete cylinders, the mass allowed to set, and in all cases the concrete would split apart from the joint when tested. One of my superintendents found himself unable to believe in the superiority of the new joint, and I had therefore a slab, about $4' \times 4'$, set aside, letting him finish one half by any method desired, and reserving the other half for my acid joint. Try as he would, he never succeeded in making an unbreakable joint. A reward of ten dollars was promised to any man on the job who could separate the finish from the base on the half treated with acid, and while many of the men availed themselves of the opportunity, the reward is as yet unearned.

The Pacific Coast Borax Co.'s building, at Bayonne, N. J., erected in 1897–98, in a measure marks the closing of the old-time construction of reinforced concrete buildings, constructed more or less in imitation of brick or stone buildings, with comparatively small windows set in walls (Figure 11). This building, however, occasioned the discovery of an important fact, that of the greatly improved fire-resistance of concrete mixed with salt. Before that time, salt had been known and used as a frost preventative, and as this building was constructed in the winter, I desired to use salt. I had some doubts as to the strength of concrete so made, and I also anticipated some trouble with efflorescence. A number of test cubes were made with salt, some mixed by hand, others by mill, and to my surprise I found that the hand-mixed specimens showed efflorescence while the machine-mixed specimens did not. I am unable to explain this difference. As to the strength, I found it was not impaired by the salt, when salt to the extent of one to five per cent. of the weight of the cement was added; I also found that the specimens without salt showed air or hair cracks, while those with salt did not. It now occurred to me that salt might have the same or similar effect on concrete that it has on clay; it is well known how clay pipes, etc., are glazed by being burned with salt. Test cubes with and without salt were heated in

FIGURE 11. — PACIFIC COAST BORAX WORKS, BAYONNE, N. J.

In Front, the original building of the old type. In Rear, a more recent addition of the modern type

Ransome & Smith Co., Contractors

the boiler furnace to a red heat, and then plunged into cold water; the specimens without salt had a soft surface easily picked to pieces with the bare fingers, while those with salt were intact, and the compressive strength appeared to be unchanged. I had then the belief that different brands of cement were affected in different ways by the addition of salt, but I have so far never found a Portland Cement that was injured in the least by addition of the quantities indicated.

Of other additions to Portland Cement with which I have experimented I must mention lime and clay. The former addition is so liable to abuse that I have largely abandoned it, except for the construction of waterproof tanks, and even then it is not indispensable. The fact seems to be that an addition of from three to five per cent. of slacked lime is beneficial when added as "milk of lime," using the limey water for mixing instead of plain water; the trouble arises as soon as lumps of lime putty, however small, find their way into the concrete, or when the amount exceeds five per cent. In cases where the concrete for this or other reasons contains free lime, I have sometimes hastened the setting by giving an artificial supply of carbonic acid; usually supplying heat to the concrete at the same time. For the details of this see my patent No. 652,732 (1900).

As to an addition of clay I have found that from two to three per cent. in the aggregate are beneficial rather than detrimental, and such moderate amounts help greatly to render the concrete waterproof. My attention was first called to this matter when conditions compelled me to use Niagara Gravel for some works in Buffalo in about 1892, and I declined at first to use this material as it evidently contained considerable quantities of clay. Inspection of concrete work made by other parties soon convinced me that the gravel in question was excellent material, and earlier tests by Mr. Clarke of Boston fully corroborated my own observations on this point. I do not wish to go on record as stating that all clay is beneficial, and if the grains of sand are coated with a film of clay, I am convinced that it has a most dangerous effect.

In the years between 1900 and 1902 I developed a radical departure in the exterior construction of reinforced concrete factory buildings, consisting mainly in the extension of the floor plate or slab over the exterior columns, forming a belt course on the outside of the building. Between the exterior piers, upward and downward extensions were added; the former to be added after the next floor had been constructed, and the latter forming an integral portion of the floor proper. This innovation forms the subject matter of my patent No. 694,580 (1902), Figure 12, according to which a very large number of buildings have been erected throughout the country.

The principal advantages of this construction as compared

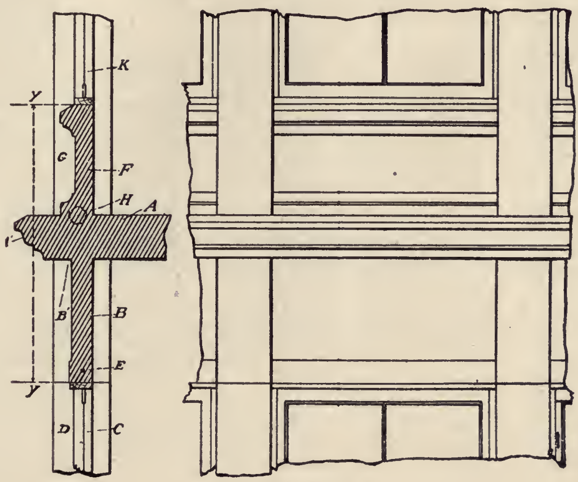


FIGURE 12.

with the old solid concrete walls are, aside from the purely technical ones, that large window areas are easily made possible, that the curtain walls are utilized as carrying members, and the shrinkage of the walls is taken care of by means of the expansion joints existing at each end of each curtain wall, the same being recessed into the sides of the piers. That this construction affords great economy will be evident from the fact that all the curtain walls may be cast with a few forms only, the several forms being usually removed in twenty-four to forty-eight hours after pouring.

The first building erected under this patent was the Kelly & Jones Co.'s machine shop at Greenburg, Pa., 60′ × 300′, four stories high (1903–4), Figure 13, built by the Ransome &

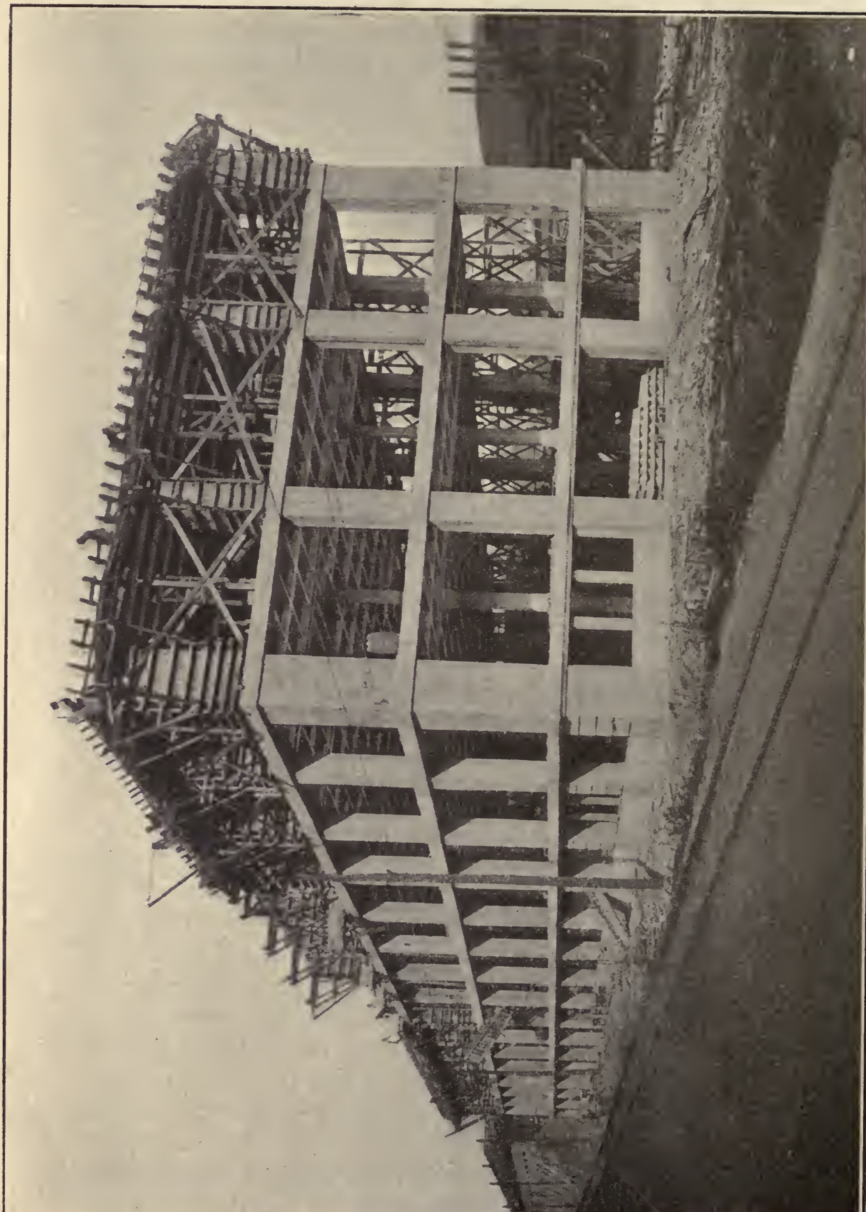


FIGURE 13. — MACHINE SHOP, 300 feet x 60 feet. KELLY & JONES CO., GREENSBURG, PA.
Ransome & [illegible] Contractors

Smith Co., and followed by many more, chief amongst which I mention the machine shop for the United Shoe Machinery Co., at Beverly, Mass., aggregating about sixteen acres of floor space (the recent additions comprising about four acres were built under the Unit System), and the Foster-Armstrong Co's Piano Factory at East Rochester, N. Y., including a dozen or more large buildings.

From this time also dates my invention of the coil joint for uniting reinforcing bars, consisting in an open metallic coil surrounding the lapping ends of the bars to be joined; No. 694,576 (1902), Figure 14. It is particularly well adapted for deformed and especially twisted bars, because the initial sliding of the bars cannot take place without driving out a wedge of the surrounding concrete, and this is effectively prevented by the coil. Wherever beams have been built with bars joined on this principle the results have been satisfactory, and the much later tests by Professor Mörsch fully substantiate everything claimed for the coil joint. (See Trautwine: Concrete, 1909, p. 1174, where plain bars only have been used.)

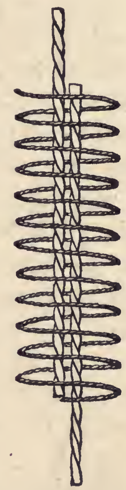


FIGURE 14.

During all this time I have given considerable study to the proper design of the falsework, realizing in common with other concrete men that the handling of the forms in many cases meant the difference between loss and profit. The standardization of the forms and their repeated use is one way of approaching this problem, and with a standard layout there is no question but that investment in molds of a more permanent nature is a paying proposition. More information in regard to this may be found in a later chapter on forms; suffice it here to say that I have made "coreboxes" for one building and used them there four times, shipped them to another building and used them seven times, then again shipped them and used them four times, and finally shipped them once more and used them, but on the last job the repairs were so expensive that the profit was doubtful. I am now convinced that the final solution of the questions pertaining to economical construction must be found along other lines, and I have had sufficient experience with " Unit Construction " to warrant the statement that great economy and better workmanship, as well as quicker work, is thus obtained. From a first attempt in 1905, and subsequent experience, I have evolved a system adapted to buildings with many stories, known as " The Ransome System of Unit Construction," which has been extensively used and is now being used on work of considerable dimensions. (Patents No. 694,577, 1902, and 918,699, 1909.)

Not very long ago, a patent was granted for " wet mixed " concrete to a Western gentleman, and this brings back to memory the historical fact that wet mixture has been known from the earliest days of the art. Thus, Coignet's early patents (1869) speak of it and recommend it, but, nevertheless, dry concrete rammed in was in general favor. Now, one of the products of the old Ransome Stone Company was porous filter stones, made under the old process, and I was very much interested in making similar stones of Portland Cement concrete. Owing to the lack of uniformity of the concrete stones, I never made a success of this, but I did find that in order to make the concrete sufficiently porous for the purpose, I had to use a dry mix. Reversely, in making ornamental stones, I always had better results with wet mixtures, especially for the facing.

It is believed that the argument in favor of the dry mix was based upon the fact, known as early as 1890, that dry mixed mortar rammed hard into the briquette molds gave higher strength in the tensile tests than wet mixture. The arching effect of the stone in the concrete was disregarded. Personally I was confirmed in my observations by Bamber's tests which ingeniously proved the fallacy of the arguments in favor of the dry mix, and showed the greater density of a wet mix: A dry batch was made, and rammed thoroughly into a mold $2' \times 2' \times 2'$ so as to fill it level full. The mixing platform was now cleaned, and the contents of the box dumped out on the mixing board, thoroughly remixed with enough water to make a " wet " mix, and then replaced in the box. But it now proved that the box lacked 2″ in being full, so that the greater compactness or density of the wet mix was proved. Other tests have proved the superior strength of wet concrete, and that the densest mixture is also as a rule the strongest; as to the permanency I have had occasion to compare dry and wet mixed concrete after they had been in place for many years, and found the wet mixture much harder than the dry placed concrete.

From my long and varied experience with concrete I desire to state that I have found no agency which actually injured old well-made concrete properly proportioned, except acid. Such items as sewage and oils have had no influence, neither have I found that the gases from the salamanders injure the setting concrete.

In closing this contribution to the history of Reinforced Concrete, I cannot help but marvel at the enormous growth of the concrete industry during the last fifty years, and especially of the reinforced concrete industry in its less than thirty years of actual use. I venture to predict that the next thirty years will see even greater advancements, but I would also ask the younger men in the profession to remember that real knowledge and everlasting care are necessary, so that the reinforced concrete industry in the future may proceed without setbacks from accidents caused by neglect or greed.

CHAPTER II

BASIC PATENTS FOR INVENTIONS RELATING TO REINFORCED CONCRETE, AND A SHORT SURVEY OF THE EARLY HISTORY OF THE ART

BY ALEXIS SAURBREY

REINFORCED concrete as used today may be said to have arisen from the following basic inventions: (1) A combination of a plastic material adapted to harden, with a metallic strengthening device, the word " plastic " being here used in its widest sense so as to include masonry laid with a plastic mortar. (2) That, in a combination of this kind, the concrete, or plastic material, must carry whatever compressive stresses act in the structure, and the metal the tensile stresses. (3) That therefore the concrete and the steel have a tendency to separate, so that "bond" or "anchorage" must be provided, either locally in certain parts of the structure, or continuously along the length of the metal reinforcement. (4) That, in addition to a main, or directly tensile reinforcement, a secondary, transverse reinforcement is desirable and beneficial, whether this transverse reinforcement is made from separate bars, or some of the main bars are arranged in a peculiar manner to gain the desired effect. (5) That a compression member may be strengthened by longitudinal as well as by transverse reinforcement, or both. (6) That a multitude of various uses may be found for the compound material, each requiring a special combination.

To trace back into remote antiquity the use of metal in combination with brick work is beyond the scope of this book; suffice it to say that the Romans are sometimes credited with the first use of such constructions: it is said that a tomb has been found in which the roof consisted of a concrete slab with bronze rods embedded, crossing each other lattice-wise. This construction dates a hundred years or more B.C.[1] It appears

[1] "Reinforced Concrete," compiled by James Tozer & Son, Limited, Birkenhead, England.

that on the authority of ancient writers, the prevailing method of judging the quality of lime for setting purposes was by observing the hardness and color of the original stone, the harder and whiter varieties being preferred, and that this method was in general use for a score of centuries or more, until the more modern method of learning by experiment and investigation of the facts was first applied to the subject by Smeaton in England, in or soon after the year 1756. Smeaton is credited with the discovery, as a result of actual chemical analysis, that the real cause of the setting of limes and cements consisted in a combination of clay with lime.

The use of the English natural cement, commonly called "Roman Cement," was discovered by Parker in 1796, who in that year took out a British patent, No. 2120, for a cement or tarrass to be used in aquatic or other buildings and stucco work. The use of the word "Portland Cement" first occurs in the specification of a patent granted in 1824 to Joseph Aspdin, of Leeds, England, No. 5022, owing its name to its resemblance to Portland Stone, and this discovery formed the basis of considerable manufacturing operations after the establishment of a factory at Wakefield in 1825.

The Period of Discovery. Although in 1847 three or four cement mills manufacturing artificial cement were operating in England, the use of the new product was limited, until definite methods of determining the commercial value of the product were developed. Amongst the engineers who made reliable and scientific observations in this field, John Grant, subsequently knighted for his eminent ability as engineer, must be mentioned in the first line, as well as General C. W. Pasley, whose book on "Limes and Calcareous Cements" was a standard work in its day (first edition in 1838, second in 1847). From this work we learn how the relative merits of the various cements were sometimes tested by building a row of bricks out from the face of a wall (Figure 15), as many as twenty-nine or thirty bricks having been stuck out in this manner in one day, and thirty-three bricks in thirty-three days, before the bricks fell. The famous semi-arches constructed by Sir M. J. Brunel, the builder of the Thames Tunnel, were undoubtedly conceived in this same spirit, and these arches are all the more interesting as they give the first known rational application of the principle of strengthening masonry by means of tension rods. The arches are shown in Figure 16; the work was four years in building, having been added to from time to time, and it fell on January 31, 1838, as a result of a cave-in in an adjoining excavation. The long arch

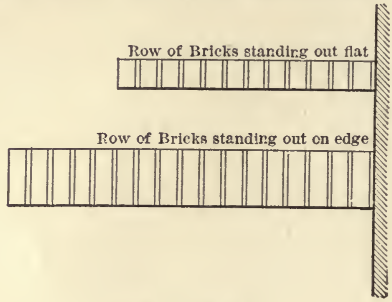


FIGURE 15.

was 60 ft., and the other about 37 ft. long, the latter being loaded at its extremity with a weight of 62,700 lbs., and this remarkable result was obtained by the introduction of wooden lath and hoop-iron bonding strips inserted in the joints of the brick work. "This ingenious arrangement of Mr. Brunel will

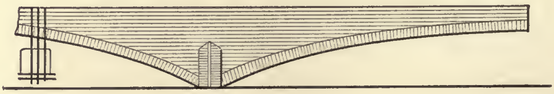


FIGURE 16.

probably be found hereafter of great value in practical architecture," Pasley says, and time has shown that he was correct in his prediction.

Brunel also built a brick beam 25′ 7″ long, reinforced with strips of hoop-iron; this beam was broken in 1836 under a load of 27,025 lbs. A similar experiment was made at the Francis Cement Factory at Vauxhall, the dimensions of the brick beam being 4′ 9″ deep by 22½″ wide, reinforced with fifteen pieces of 1¼″ hoop-iron in the bottom portion. The span was 21′ 4″ in the clear, and the beam was broken under a load of 50,652 lbs.

Pasley had now several such beams made, one laid in neat cement without irons, one exactly similar, but reinforced with five longitudinal irons, and a third beam similar to the second one, but laid in 1 : 3 lime mortar. The result showed the superiority of the reinforced beam laid in cement, for the first beam carried only 498 lbs., while the second carried 4523 lbs., and the third failed by sliding of the irons under a load of only 742 lbs.

As early as 1832, Ranger took out a British patent, No. 6341, for making certain kinds of mortar with hot water, and this invention was used in the first known case of modern engineering work of any consequence executed entirely in concrete, viz., a dock at Woolwich dockyard (1835) and sea-walls at Woolwich and Chatham. The floor of the dock was a failure, but the sea-wall at Woolwich was standing in perfect condition in 1879. "Ranger's artificial stone" became well known in England, but it was soon discovered that cold water was quite sufficient for making good concrete.

A short list of some early English patents may be of interest:
Wilkinson, 1854, No. 2293 (Figure 17).

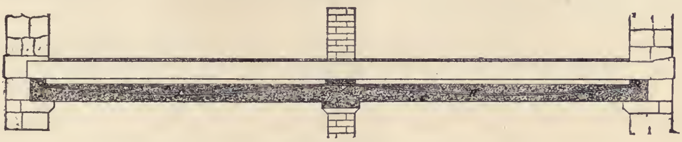


FIGURE 17.

"The wire rope *H, H*, is secured at its extremities at each line of support by imbedding it in the mixture or concrete while in a soft state, and forming the ends into loops, or by opening out the strands and hirling them in various directions, which renders it so secure as not to be drawn out under any force short of the breaking weight of the rope. For ordinary dwelling houses I propose placing such wire ropes about nine inches apart, and to have a full depth of floor of one-sixteenth the span."
Dennet, 1857, No. 685 (Figure 18).

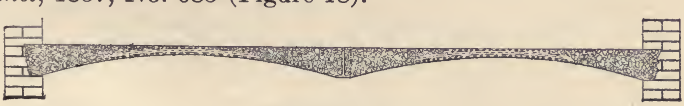


FIGURE 18.

Proposes to strengthen his arches with lamina of wood or iron.
Bunnet, 1858, No. 1292 (Figure 19).

Uses iron tie rods and metal abutment plates for his arches of hollow blocks.

Parkes, 1863, No. 317.

Proposes an iron bond consisting of a band or strip of iron with transverse teeth, ridges, ribs or projections pressed out of the solid, raised at intervals on each side of the strip for the whole width thereof. He employs two rollers, with suitable indentations, for the manufacture.

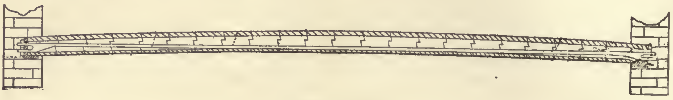


FIGURE 19.

Ransome (*Fk.*) 1865, No. 1337.

Molds slabs of artificial stone around pieces of hoop-iron on edge running from end to end, so that the hard concrete prevents the irons from buckling under load.

Scott, 1867, No. 452 (Figure 20).

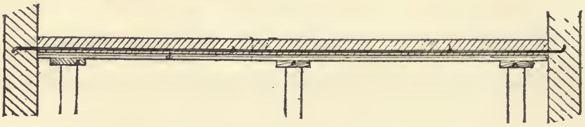


FIGURE 20.

Proposes to dispense with the use of ordinary joists and to make use of wrought-iron tie-rods extending from wall to wall. "The floor becomes one solid beam, having the tie-rods and hoop-iron in combination with the concrete to take the tensile strain, and the concrete to take the compressive action resulting from the weight of the floor."

Lythgoe & Thornton, 1868, No. 640 (Figure 21).

FIGURE 21.

"The method of constructing floors with bars of ⊥-iron and concrete as shown."

Johnson (*Coignet*) 1869, No. 884 (Figure 22).

An invention relating to the facing of concrete blocks with cast-iron or steel protecting plates, to be used as street curbs, etc.

Gedge (*Monier*) 1870, No. 1999 (Figure 23).

"In short the iron is the skeleton and the cement its covering."

Tall, 1871, No. 1001 (Figure 24).

Iron hooping, wirework, or netting are interlocked between

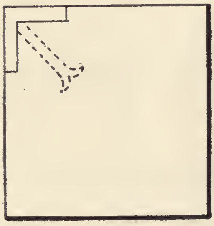


FIGURE 22.

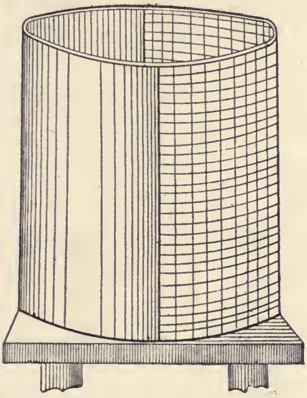


FIGURE 23.

the lateral cross bars, and form a close lattice or basketwork. Portland Cement stucco is applied.

Brannon, 1871, No. 2703 (Figure 25).

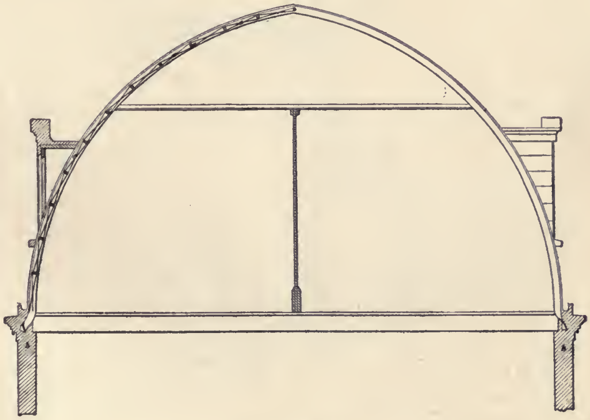


FIGURE 24.

"Wirework embedded in concrete, to give cohesive strength against transverse and tensile strains."

Hyatt, 1871, No. 3124 (Figure 26).

"The peculiar construction of floor which I designate an 'all-beam' floor, composed of a number of separate tubes laid side by side."

Turner, 1872, No. 1396.

On the iron beams "I strain my wire from the plates in the walls; these wires are intended to supersede the use of floor joists of wood, and will form beds for my concrete floors, and also answer on the underside instead of laths for the plastered ceilings, which work of plastering may be carried on at the same time as the laying on of the floors in concrete."

Emmens, 1872, No. 2451 (Figure 27).

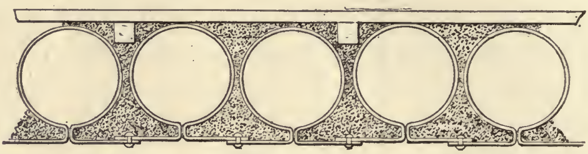


Figure 26.

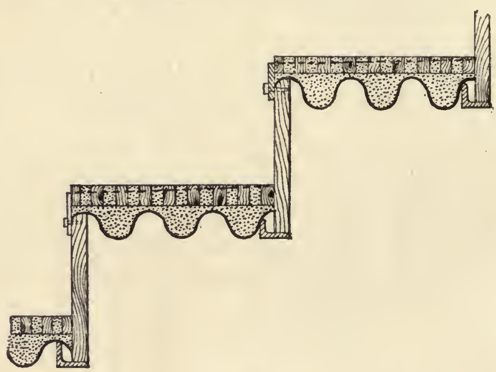


Figure 25.

Figure 27.

"The employment of sheets of corrugated iron as foundation for roadways, paths, steps, and flooring."

Lish, 1873, No. 1621 (Figures 28, 29).

The drawing shows a sectional view of a floor and girder of concrete with tension rods embedded therein, as indicated by the dotted lines.

Hyatt, 1873, No. 3684.

Asbestos combined with perforated, corrugated sheet metal or with crimped sheet metal or upon a hollow grate bar system.

Coddington, 1873, No. 1004 (Figure 30).

The figure shows a water pipe or tube, Ç being the cemented material, E the interwoven metal.

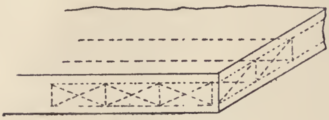


FIGURE 28.

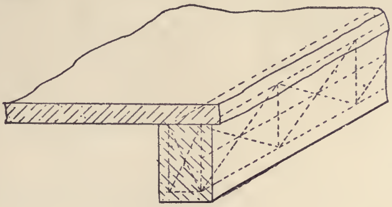


FIGURE 29.

Hyatt, 1873, No. 3381.

"The system or mode of forming cellular or honeycomb structures by connecting together single cell blocks by means

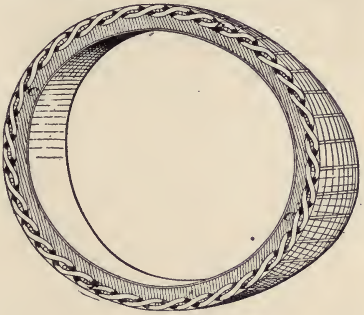


FIGURE 30.

of tie-rods or crimped blades of metal, with or without additional straight tie-rods."

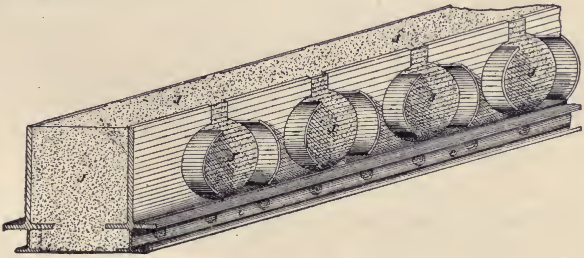


FIGURE 31.

Hyatt, 1874, No. 2550 (Figure 31).

"I form the tie in a way which gives it power to grip and hold the foreign material in a manner and by a method which brings the load and consequent strain upon the tie at the same instant it is felt by the concrete or foreign material, by which means the tensile and compressive forces act in harmony with each other."

Hyatt, 1874, No. 1715 (Figure 32).

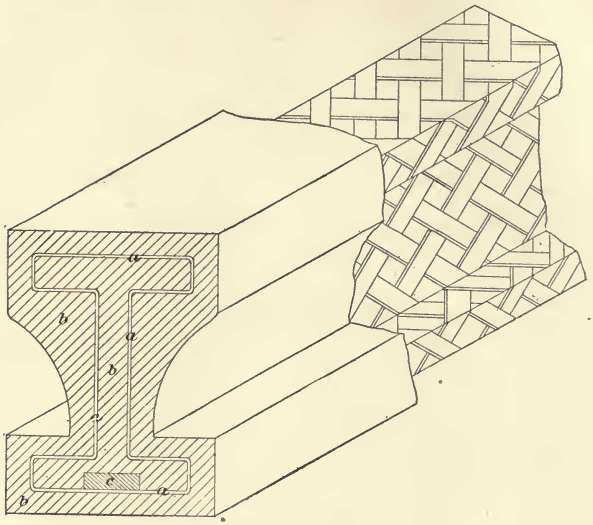


FIGURE 32.

" Making hollow metal beams of interlaced lattice or open-work, as the holder of a tie-rod, to connect the same with concrete or equivalent material."

Edwards, 1891, No. 2941 (Figure 33).
1892, No. 1415 (Figure 34).
1894, No. 15,466 (Figure 35).

FIGURE 33.

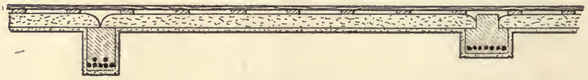


FIGURE 34.

Edwards' patents show a remarkable insight into the nature of reinforced concrete construction. It is proposed to cast the slabs separately and set them when hard, owing to the great cost of the centering; the bending up of the principal tension rods is described at great length, and stress is laid upon the benefit of many small rather than fewer but larger rods. The

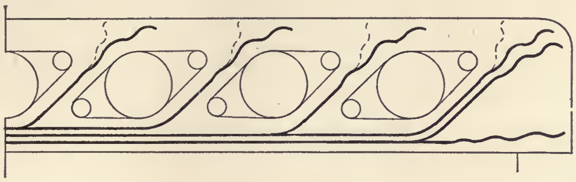


FIGURE 35.

importance of preventing sliding of the reinforcement is shown, and it is described how the beams may be pierced by openings in much the same manner as done under the Visintini System. The benefits as well as troubles arising from the fixing of the ends of the beams into the walls are perfectly understood, and the entire argument advanced is illustrated by tests (by Kirkaldy).

While in *England* the new construction made but scant headway, a considerable activity took place in *Germany*, where the Monier Patents were bought and exploited by G. A. Wayss, and where M. Koenen advanced the first rational method of calculation in 1886. The "straight line formula" was fully discussed by Koenen in "Centralblatt der Bauvervaltung," May 14, 1902, and is to this day the commonly accepted standard.

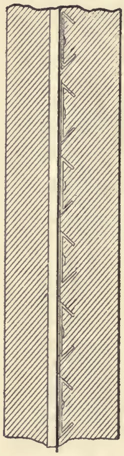
In *Holland*, the first ribbed floors were erected in 1886 in connection with the Public Library in Amsterdam.

In *France*, it seems that Monier's first patent was taken out in 1867, but it has been intimated that he had knowledge of the earlier patent granted to Lambot, who had made a reinforced concrete boat of small dimensions in 1855. This boat is said to be in existence today. Monier's efforts toward the introduction of his inventions were not very successful, partly perhaps because he failed to realize the necessity of placing the reinforcement near the bottom; it is told that when Wayss showed him slabs so reinforced, Monier severely criticized this arrangement, and abruptly ended the argument by exclaiming, "Who is the inventor, you or I?"[1] As a matter of fact, little

[1] Suenson: Jaernbeton, P. 5.

was done in building construction until 1892, when Hennebique and Coignet took the reinforced concrete construction up with great success, each introducing his own system.

In the *United States*, the first indication of anything approaching reinforced concrete may be found in a patent granted to P. Summer, 1844, No. 3566 (Figure 36), for a metal lathing, which was still further improved by J. B. Cornell in 1859, No. 22,939 (Figure 37). At this early date, a number of patents



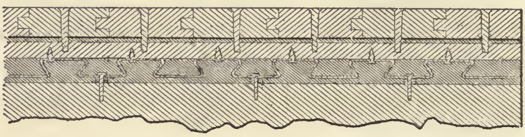


Figure 37.

Figure 36.

for cement pipes were granted, as to R. B. Stevenson, 1854, No. 11,814, for a combination of a pipe of sheet-metal and an exterior coating of hydraulic-cement mortar of "requisite thickness for strength." In the Wyckoff patent, No. 32,100, of 1861, the interior pipe is of wood wound with wire of iron or other metal; in the Knight Patent, No. 32,298, of the same year, a metal tube is disposed "intermediate between the inner and outer surfaces" of a cement pipe. In 1868, A. P. Stephens took out a patent, No. 78,336, on a similar pipe, in which the strengthening tube was made of corrugated iron; in 1872, Patent No. 127,438, the tube was changed to a spirally formed sheet metal tube, and in the same year J. A. Middleton, Patent No. 133,875, proposed to strengthen his cement pipes by a layer of wirecloth embedded in the cement, thus combining what we now consider the essential elements of a reinforced concrete pipe (Figure 38).

The first reinforced concrete wall-patent appears to be one granted to S. T. Fowler, in 1860, No. 28,069, where the concrete wall is to be strengthened with vertical and horizontal timbers, to be buried in the concrete; a more rational construction is

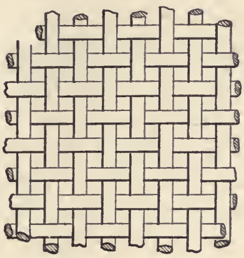


FIGURE 38.

proposed in 1862, No. 37,134, by G. H. Johnson, for grain-bins: "a new construction formed of brick-work tied together by plates and rods of iron." In 1869, No. 87,569, G. H. Johnson

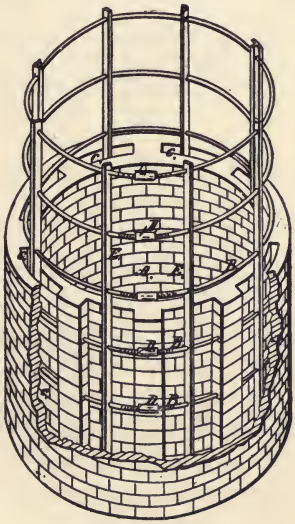


FIGURE 39.

improved this construction, using "horizontal annular tension-bars . . . the ends of each bar being so united as that it shall form an endless, unbroken band . . . in the combination . . . with . . . vertical connecting-rods so as to form a metallic frame within the walls of the structure." This invention (Figure 39) was not the only important improvement of that day; in 1868, C. Williams, No. 75,098, invented the metal lattice-reinforcement for concrete walls. The lattice-work was built up by riveting the slats together (Figure 40).

The first use of concrete in columns must be conceded to W. H. Wood who, in 1862, No. 36,747, patented an improvement in piers and bridges. The invention consists in the use of hollow cast-iron columns filled with concrete or cement, and supported on wooden spiles below the surface of the bed of the river. The first ceiling was proposed by J. Gilbert, 1867, No. 64,659. This patent shows corrugated iron plates filled with

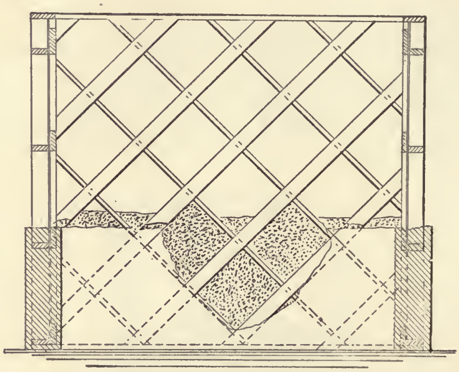


FIGURE 40.

concrete, the concrete to extend an inch or so above the top of the corrugation (Figure 41). His solution of the problem "self-centering reinforcement" is not very inferior to those proposed by more recent inventors. Thus we see that around the year 1870 the combination of masonry of various kinds with a strengthening metal work was quite well known. The patent, No. 88,547, granted to F. Coignet, a Frenchman, in 1869, states the general principles very clearly: " In the body of artificial stones": "skeletons or metallic framework, linked or arranged so as to strengthen the same." This is the whole science of reinforced concrete construction in few words. As an example, he proposes to use a cylindrical web of small rod-iron or wire in combination with a cement envelope, for the purpose of resisting the interior pressure in pipes, as well as T- or

L-irons for other purposes. The series of patents granted to Coignet in 1869 deserve more than usual attention, as they contain much good advice of value to engineers; they are No. 88,545, 88,546, 88,547, 88,548, and 88,549.

FIGURE 41.

The brick arch with abutment-shoe and tension bar between abutments was invented by C. Henderson, in 1871, No. 113,881 (Figure 42); the brick arch reinforced on the cantilever principle was invented by F. Alsip, No. 120,608, in the same year. It is not clear from the description whether Alsip really con-

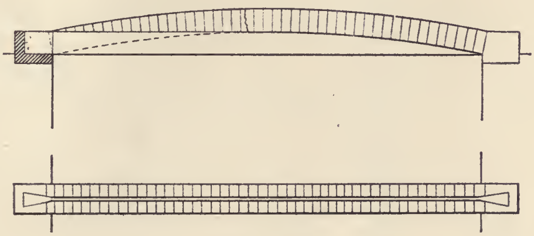


FIGURE 42.

sidered his invention as a cantilever construction, but the fact remains that all the essential elements of a cantilever are present in this patent. A very interesting patent No. 122,498 is the one granted to W. H. Smith, in 1872, for a concrete pavement. On soft ground, the arched pavement is intended to be self-supporting. Tie-rods are then carried under the pavement from

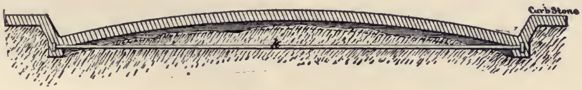


FIGURE 43.

curb to curb, or "chords may be embedded in the composition to operate in lieu of abutments to the arch." In the drawing, the tension rod is shown provided with a large button on the end, evidently for the purpose of preventing slipping of the bar (Figure 43).

The patent issued to Sisson and Wetmore, in 1872, No. 124,453 (Figure 44), shows "a combination of trussed and untrussed frames of light bar-iron to form skeleton wall-posts, girders, etc., in combination with a filling of beton or other suitable concrete, to be poured in a state more or less liquid. Our object is to have the beton and iron frames furnish mutual support and protection to each other." Considered as a beam,

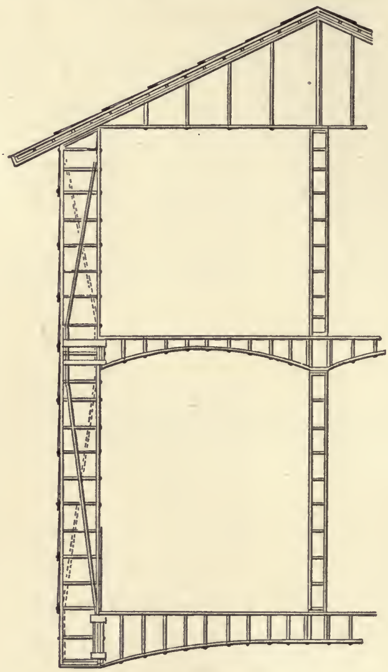


FIGURE 44.

the wall-post of this patent exhibits many of the essential features of present day practice: the top bar extending from one span to another, the trussed bar bent up over the support, the horizontal lacing of the verticals and the vertical lacing of the horizontals, etc.

But generally speaking, the reinforced brick-arch continues to hold the interest of the inventors. In 1872, P. H. Jackson received a patent, No. 126,396 (Figure 45), for a peculiar construction of abutment-casting to be used in connection with reinforced arches, and in 1873, No. 137,345, N. Cheney proposes to make the tension reinforcement of light wires placed close together and interwoven with cross-wires, to serve the additional purpose of a metallic lathing. The earthquake-proof house invented by D. L. Emerson, in 1873, No. 137,833, calls

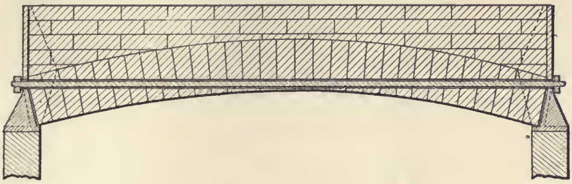


FIGURE 45.

for vertical rods or plates in the walls, and anchors passing through them, the plates and anchors being connected with strap iron. In the same year J. W. Basset, No. 138,118 (Figure 46) shows a construction of individual plaster slabs with a metallic trellis work within, the ends of which extend beyond the block, for the purpose of locking the various blocks together.

While not strictly within the scope of this paper, attention is called to the patent, No. 172,641, granted to O. C. Matthews,

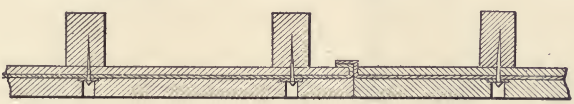


FIGURE 46.

in 1876, for a foundation, in which piles are driven and again withdrawn and the holes filled with concrete (Figure 47).

In 1878, T. Hyatt, No. 206,112, ended the "period of discovery" and put the theory of reinforced concrete construction on a rational basis, and at the same time received a patent of remarkably broad scope, covering practically the entire field of reinforced concrete and masonry construction. The general purport of this invention is set forth in a volume entitled "An account of some experiments with Portland Cement concrete, combined with iron," of which a copy was deposited in the library of the Patent Office, but which was otherwise designed for private circulation. Hyatt appears to be the first to state specifically that the steel must be able to resist sufficient tensile stress to balance the compressive stresses on the concrete, that all metal may be dispensed with save the tension rod only, that both baked bricks and concrete possess in themselves cohesive power and strength sufficient to perform the functions ordinarily performed by the metallic web. He realizes the value of deformed bars and says: " I prefer to use metal specially rolled for the purposes, with bosses or raised portions formed upon

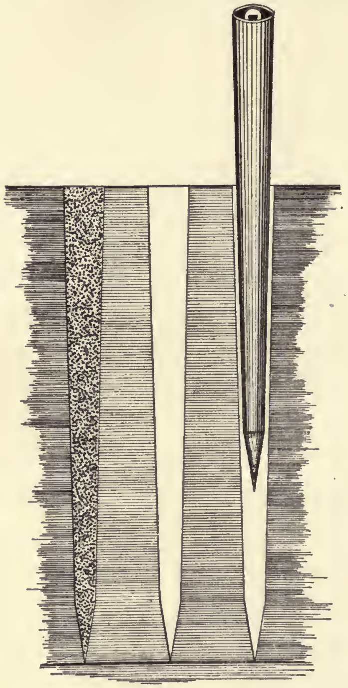


FIGURE 47.

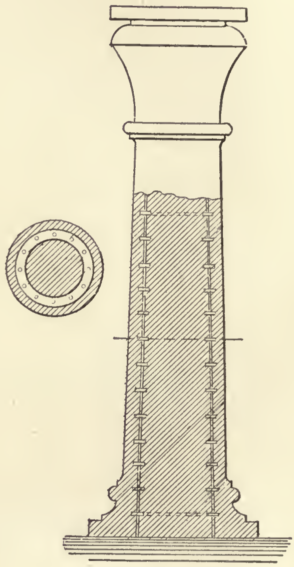


FIGURE 48.

the flat faces of the metal. When I make use of common bar or hoop iron, I stud the slips with pins; or I make use of several blades threaded upon wires, as represented by Figure 1." In the book mentioned above, he laid down the results of his experiments which led him to bend some of the bars up, and also to use a rigidly attached separate " shear member." The analysis is very complete, both in his book and in his patent specification. He reinforces his columns with longitudinals or horizontal hoops, as the case may require, or both. He says: "In constructing the columns or piers wholly of concrete, I make the structure solid, the concrete then bearing the load, and, giving way under compression, would naturally incline to yield in the first place, not from absolute crush of the materials, but from want of sufficient tensile resistance at the circumference of the column. But this tendency being resisted by the circular ties, such a concrete could give way only by the crush of its particles." In short, the whole theory of hooped columns. The only difference is that Considère prefers the use of spirally wound reinforcement, while Hyatt uses the individual bands (Figure 48).

To what extent Hyatt was familiar with Pasley's tests, if at all, we do not know; in his book of 1877 he gives a brief account of the history of fireproof construction, but gives no reference whatever to the tests just mentioned. It appears that he had a test made in September, 1855, in New York, under the general supervision of Mr. R. G. Hatfield; the beam was about 9″ square, and had a tie-rod passing through holes made for the purpose in the bottoms of the bricks. More important tests were made by Kirkaldy in London, from 1874 to 1877, on beams made by Hyatt.

The Period of Improvement. Broadly speaking, the Hyatt Patent, No. 206,112, shows and describes everything necessary

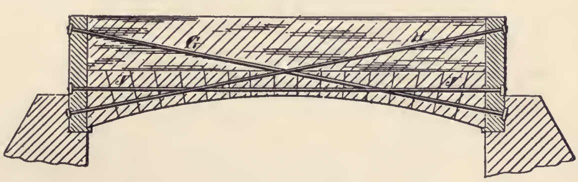


FIGURE 49.

for the practical use of reinforced concrete, and the patents of the following period are therefore mainly for improvements, many of which are due to Hyatt. Most interesting is the one granted in 1883, No. 290,886, for a concrete floor, showing not only transverse arches between the ribs, but also the use of web reinforcement in a continuous sheet along the center of the beam. In 1881 a patent, No. 237,471, was granted to S. Bissell (Figure 49) for an arch-bridge, showing diagonal straight reinforcement within the masonry, the object being to construct "an arch of limited span without causing any horizontal thrust upon the abutments." The Cubbins patent of 1883, No. 285,801, shows a circular cistern cover "of artificial stone, having a metallic band or tire" (Figure 50) or "consisting of

FIGURE 50.

a concavo-convex or arched disk . . . inclosed by a metallic band or tire." This appears to be the first slab with "circular reinforcement." "Expanded metal" was patented, No. 297,382, in 1884, by J. F. Golding: "metallic screening formed of slashed and stretched metal." The particular use to which the invention was to be put is not specified, and at first it was used exclusively as a metal lath. Its use as reinforcement for structural concrete is of much later date (Figure 51).

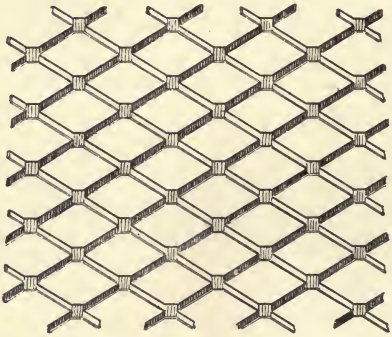


FIGURE 51.

A number of interesting patents are granted at various dates to P. H. Jackson. The first, No. 302,338, in 1884, is not of interest in this connection; it shows principally the usual tie-rod construction in a brick arch. But the following year, 1885, he took out a patent, No. 314,677, showing, for the first time, the bent-up or "trussed" arrangement of reinforcement (Figure 52); the bars are carried to the support where they are anchored by means of nuts. The concrete and its reinforcement rest upon corrugated iron plates, and the bars may be secured or not at intervals to the bottom of the corrugated plates. Another patent, No. 320,066, of the same year, shows the reinforcement continued into the adjacent bay and there hooked

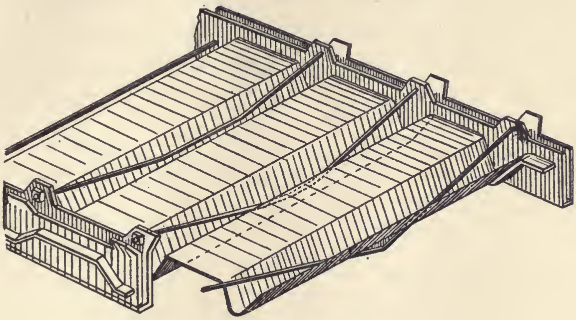


FIGURE 52.

over the tops of the I-beams (Figure 53), which here have the function of the main girders. The patent, No. 339,296, of 1886, specifies an expansion joint in the construction of a reinforced concrete arch; evidently the troubles caused by expansion and shrinkage were well known at this early date. Two patents, Nos. 366,839 and 366,840, were taken out in 1887 for "series

FIGURE 53.

of arches composed of concrete, and a longitudinal tie on which the footings of the said arches are supported and to which they are fastened," and a construction of arches with longitudinal reinforcement near the bottom; these arches rest on one side on the front girder of the building, on the other side upon the area wall. Also from that year date the following patents: two, Nos. 367,343 and 370,625, showing the application of dovetailed corrugated plates filled with concrete (Figure 54), and

FIGURE 54.

three, Nos. 371,843, 371,844, and 371,845, showing the use of I-beam reinforcement in the bottom of the beam, as well as compression-reinforcement in the top (Figure 55). The reissued, R10,921, and the original patent, No. 375,999, issued in 1888, may be noted in passing.

When we consider the state of the art as it appears from the patents mentioned above, the Monier patent of 1884, No. 302,664 (Figure 56), cannot be called much of an improvement.

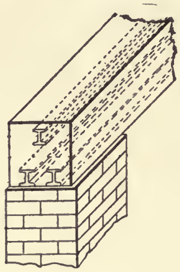


FIGURE 55.

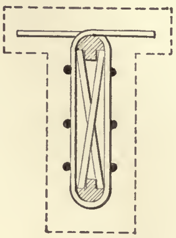


FIGURE 56.

Nevertheless, the name Monier was for many years synonymous with "reinforced concrete," at least in Europe, where the Monier patents were bought and greatly developed by German engineers. "My invention," he says, "relates to the use and sale of integral elements of construction of metal and concrete or mortar combined, the mortar forming the covering for a metal skeleton. This skeleton is composed of longitudinal bars or rods and transverse ribs, secured together by metal ligatures." The Monier patent, No. 486,535, of 1892, is practically nothing but a series of special designs based upon this same principle, and contains little new material. Yet a great industry was based both here and in Europe upon the Monier patents.

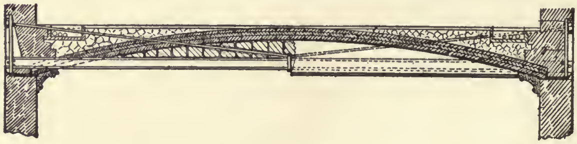


FIGURE 57.

The Ransome patents have been described in an earlier chapter and are not referred to here.

The "trussed" arrangement of the steel was, as stated above, invented by Jackson in 1885. The Gustavino patent, No. 336,048, of 1886 (Figure 57) shows the same feature, as well as the rod with a continuous curve between supports. In addition to this tie-rod which extends from wall to wall, "I may in practice use a straight tie-rod extending between wall and wall above the arch." The same year, 1886, saw the origin of another new type of construction which stands on the border between reinforced concrete and plaster work. The Rabitz construction, No. 339,211 (Figure 58), calls for a metallic skele-

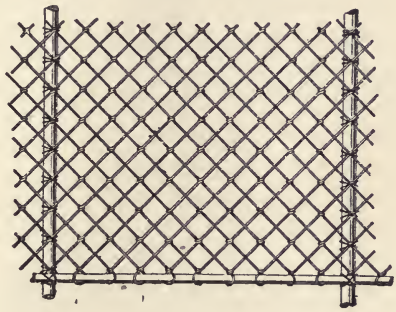


Figure 58.

ton frame of vertical rods and a reticulated metallic netting, in combination with a suitable coating of cement mortar or similar material. In the patent issued to P. M. Bruner, No. 356,703, in 1887, something approaching the U-bar (Figure 59) is shown;

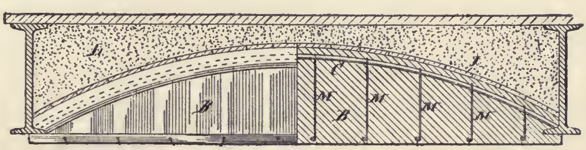


Figure 59.

although the construction would not be classed as reinforced concrete at this present time, the rods being disposed principally on the compression side, from which rods transverse ties hang down in the beam. A telegraph pole was invented by D. Wilson, No. 374,103, in 1887; it was to be composed of a skeleton frame having rods and horizontal hoops, and a coating or body of cement inclosing the frame. The same idea was patented, No. 411,360, in 1889, by O. A. Stempel, who claims a post, rail-tie, or beam, composed of "a metal frame, the filling and inclosure of imperishable material that protects said frame from the inroads of moisture and rust, and said frame arranged to protect said structure from breakage." The drawing looks somewhat like what an engineer would prepare for a column at this time (Figure 60).

Figure 60.

The patents granted to M. F. McCarthy show again "the combination (with an I-beam supporting the slab) of the wire strands extending over and drooped between the same, and the concrete filling wherein said beams and strands are embedded." This quotation is from the patent issued in 1891, No. 455,687 (Figure 61); the four patents, Nos. 520,489, 520,490, 520,491, and 520,492, issued in 1894, show various combinations and variations of the same principle. The patent issued to P. Cottancin, No. 459,944, in 1891, is for a strengthening web "characterized by the union in a reticulated fabric of a warp and a weft, each composed of a wire, band, or bar bent on itself into a sinuous or like shape." This patent forms the base for a large industry especially in France. The J. Melan patent, No. 505,054, of 1893 (Figure 62), claims "a vault or arch consisting of abutments, beams, or girders, arched ribs rigidly connected with said abutments, beams, or girders, and a filling of concrete or the like between said ribs." A number of arch-bridges have been constructed under this patent. A. L. Johnson patented, No. 550,177 (1895), a construction of floors much used at one time in the West, comprising mainly I-beams with suspension straps fastened at the tops of the beams and drooping between the beams; the straps are flat and support the concrete rib of the beam (Figure 63), upon which in turn rests the concrete slab. Another important arch-patent, No. 583,464, was granted to F. von Emperger, in 1897, for an improvement in the Melan patent described above; it consists mainly in using two ribs instead of one, each rib being placed near one surface of the concrete. Secondary members connect the top and bottom ribs (Figure 64).

Recent Patents. The idea of molding reinforced concrete members separately and afterwards erecting them in place appears to be almost as old as the art itself, and a number of

FIGURE 61.

the patents mentioned above refer to this possibility without going much into the details. In 1898, a patent, No. 606,696,

FIGURE 62.

was issued to G. B. Waite for a beam construction (Figure 65), the sole object of which is to provide members adapted to be

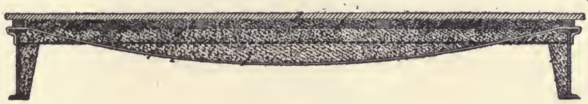


FIGURE 63.

molded in advance and erected in place after hardening. The individual sections are made of I-shape and reinforced in top

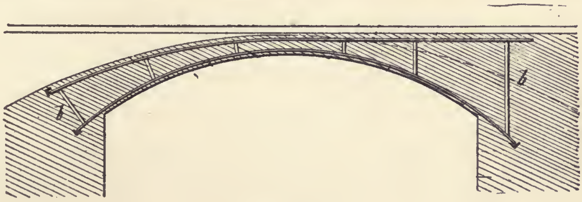


FIGURE 64.

and bottom, or in the bottom only; "shear" members of various forms are used in the beam-webs. The De Man twisted

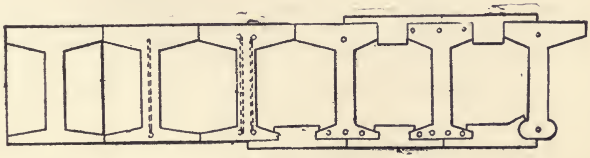


FIGURE 65.

bar was patented, No. 606,988, in the same year; it consists of "a thin flat bar having twists formed therein at intervals" (Figure 66).

The patents granted to F. Hennebique, in 1898, are three in number. The first, No. 611,907, shows the now almost univer-

FIGURE 66.

sally used combination of open, U-formed shear members with horizontal and trussed main reinforcement, with the main bars extending into the adjacent span (Figure 67). While the

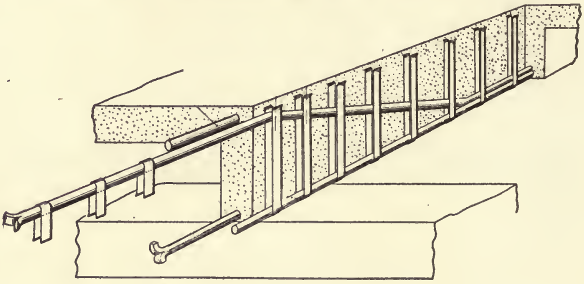


FIGURE 67.

authorities seem to disagree in regard to the value of the protection afforded by this patent, there is not the slightest reason to doubt that this construction has been of the greatest benefit to the art. The second Hennebique patent, No. 611,908, is for a system of separately molded members, claiming in substance a combination of joists and "a plurality of slabs having projecting cores embedded in said joists" (Figure 68); the word

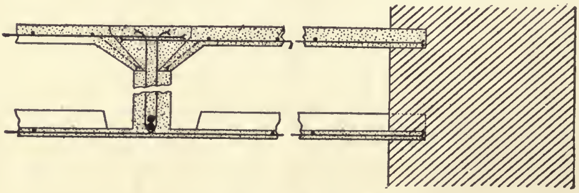


FIGURE 68.

core means here the reinforcing bar, and the slabs are placed with their ends resting upon the side-forms for the joists, so that, when concrete is poured in the joist-molds, the projecting ends are embedded in the concrete. The third patent, No. 611,909, is for a pile of reinforced concrete having grooves in two faces, so that a tight cofferdam may be made by using the piles for sheet piling, and filling in the grooves with grout. The structures erected under the Hennebique patents are numbered by the hundreds in any one of the several civilized countries.

The patent, No. 617,615, issued to E. Thacher, in 1899 (Figure 69), for an arch construction, claims the combination of the

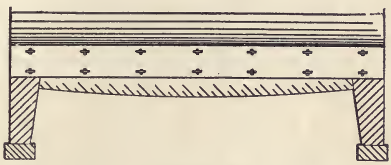


FIGURE 69.

concrete arch with its abutments, and reinforcing bars in pairs, one bar near the intrados, and one near the extrados, the two bars of each pair to be above one another, either both or only one of these bars to extend well into the abutment, and, in particular, "each bar of a pair to be independent of the other." A comparison with the Melan and v. Emperger patents is of interest, as the bars in the v. Emperger patent extend into the abutments and are placed one above the other. In the same year, 1899, a patent (Figure 70), No. 634,986, was granted to

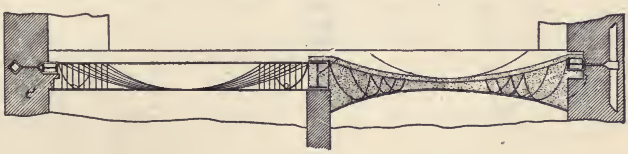
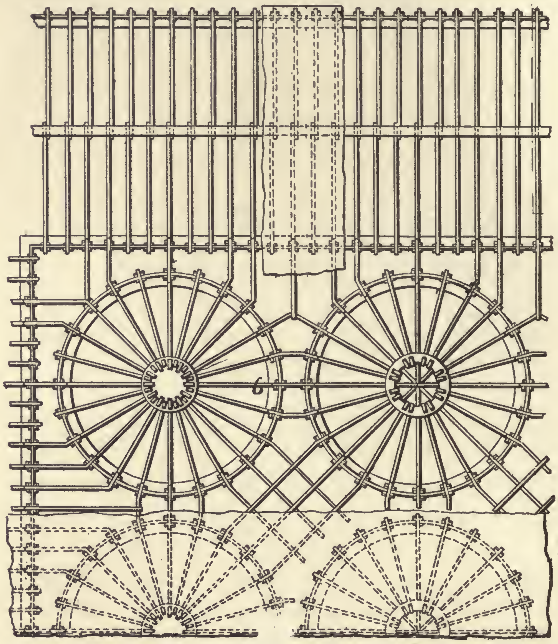


FIGURE 70.

A. Matrai for a system of wire reinforcement embodying many interesting features. One object of the construction is to unload as far as possible the middle of the supporting beam or girder, and these again are reinforced with a number of suspension cables or wires. This construction is in considerable favor in Europe. In 1900, a patent, No. 654,683, was issued to I. A. Shaler, for a construction embodying the use of longitudinal and transverse rods, the latter welded to the main bars at intervals, and in the same year, L. G. Hallberg had a patent, No. 659,967, issued for a foundation built on the principle of "circular reinforcement" (Figure 71) in combination with radial bars. The



FI[illegible] 71.

Wayss patent, No. 673,310 [illegible] 72) of 1901, is of interest, on account of the rigidly attach[illegible] ear members and other features, the purpose being to obtain similar advantages as outlined for the Hennebique patent without infringing the same;

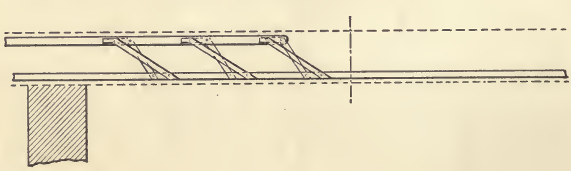


FIGURE 72.

the construction is dissimilar to Hennebique in the particular arrangement of the parts. The well-known Thacher bar was patented in 1902, No. 691,416 (Figure 73), and in the same year a patent, No. 709,794 (Figure 74), was granted to W. C. Parmley for a concrete arch construction, in which the steel is so arranged as to make the same bar pass from the tension region

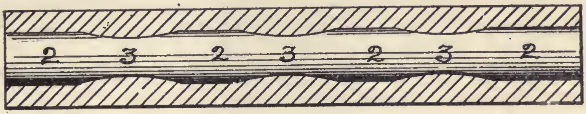


FIGURE 73.

near the intrados to the tension region near the extrados, etc. The Visintini patent, No. 735,920, of 1903, shows the peculiar type of construction known under that name; instead of the

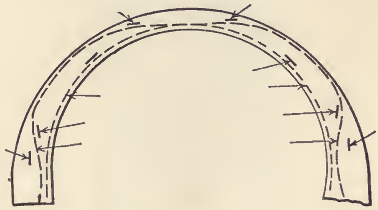


FIGURE 74.

ordinary solid beam, a lattice-girder of reinforced concrete is used. The top and bottom flanges are reinforced with longitudinal bars, and the cross-bars are embedded in the concrete

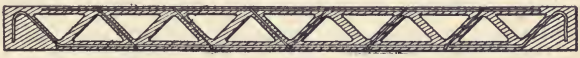


FIGURE 75.

work of the lattices (Figure 75). The Visintini beam has been used but little in this country, but abroad a large number of structures have been erected under this patent. In 1903,

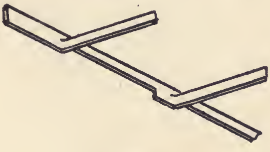


FIGURE 76.

the first Kahn patent, No. 736,602, was issued, to be followed by many more (Figure 76). The principal features are well known: The rigidly attached secondary members are manufactured in one piece with the main tension rod, then sheared loose from the main body along the greater part of the length

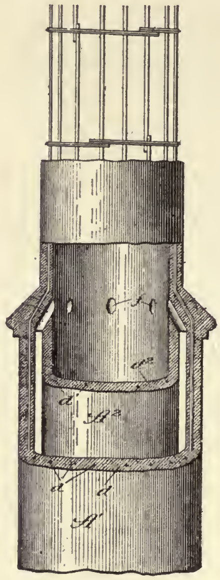


FIGURE 77.

of the rod and bent up as desired. The Weber chimney-construction was patented in 1903, No. 748,242; the lower portion of the chimney is provided with a circumferential air-space

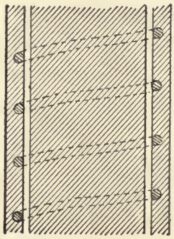


FIGURE 78.

open at its base to the outer air and leading at its upper end into the chimney flue at the base of the upper single flue (Figure 77).

A. Considère took out a patent, No. 752,523, for his well-known column construction, claiming "a solid concrete core with independent helicoidal coils of metal surrounding said core, and arranged very close together," and also the combination of these elements with separate longitudinal rods, in 1904 (Figure 78). With this patent we may consider the period of invention as coming to an end. A very large number of patents have been granted since, mostly for slight improvements, and an enumeration of all these details would be very tedious and without serious importance, although several patents of the greatest interest may be found in this great mass of dead material.

PART II

RATIONAL DESIGN OF REINFORCED CONCRETE BUILDINGS

BY ALEXIS SAURBREY

CHAPTER III

INTRODUCTION

1. Experience teaches that concrete beams may be greatly strengthened by introducing a comparatively small amount of steel within the concrete, according to certain principles of which the following is a discussion. This combination of concrete and steel is called *Reinforced Concrete;* the essential peculiarity of reinforced concrete structures is that both the concrete and the steel, if alone, would be grossly inadequate for the load which they will carry when combined; the load carrying capacity is not the sum of the individual capacities of the concrete and the steel. This general rule is not without exception, if structures like the ordinary reinforced concrete column are included; strictly speaking, only the hooped column is entitled to be classified as reinforced concrete, because in that case a small amount of steel added to the concrete changes the structural properties of the column entirely.

2. The stresses in a reinforced concrete structure are necessarily complicated. Not only is the steel entirely dissimilar in nature to the concrete which it reinforces, but the concrete itself is not homogeneous in the strictest sense of the word. Yet two cubes of large size, cut from different parts of the beam, must be assumed to be theoretically alike; — we make therefore the necessary and justified assumption that the lack of homogeneity of the concrete is of second order as compared with that of the structure as a totality: necessary, because otherwise we cannot advance any theory; justified, because the differences between the nature of steel and concrete are sufficiently large to overshadow completely the small differences which undoubtedly exist within the concrete itself.

3. Generally, the properties of reinforced concrete are known when the properties of the two materials are known; — there is

no reason for believing that the properties of either material are changed in any way by the presence of the other. It is, however, necessary to expand the limits of our research when dealing with a combination of two materials, because the properties of the combination depend primarily upon the ability of the two materials to co-operate, and only in second line upon their individual properties such as strength, elasticity, etc. This co-operative ability is of a somewhat obscure nature; — without making any attempt of explaining it, we must admit its existence. In the following it is referred to as the "bond" or the "adhesion." When this bond is broken the structure fails.

4. The purpose of design is to produce not only a structure of adequate strength, but one of equal strength in its several parts. With consistent formulas for the various elements, the allowable stresses should therefore be the same for all elements of the structure considered. Experience shows, however, that the difficulties to be overcome in the erection are different for different parts; we can readily see that a local deposit of bad concrete as large as a hand will affect a 10″ column and a 6″ floor slab in dissimilar ways. This is the reason for variable allowable stresses — in any case the purpose of fixing certain maximum stresses is to insure an ample factor of safety. Fortunately the investigation of stresses in a given beam is very much simpler under moderate loads than near ultimate failure; the coefficient of elasticity for steel E_s is a constant, and that for concrete E_c varies but slightly. For practical purposes the ratio $E_s/E_c = r$ is assumed to be a constant up to the limit of the allowable stresses. Within this same limit we assume sections plane before the load was put on to remain plane under load, and we assume proportionality between stress and deformation. The tensile strength of the concrete is entirely disregarded. None of these assumptions can be called absolutely correct; they are, however, no more inaccurate than any other set of assumptions which we would be able to suggest in our present state of knowledge; moreover, they are the simplest possible.

5. As the tensile strength of concrete is much less than its compressive strength, the principle is to utilize the available compressive resistance and use steel bars to carry the tension.

Sometimes steel is also used in compression, although with less success, the object being to limit the size of the columns and to fortify them against excentric loads. We shall see later that it is possible to construct a column in which the steel is stressed in tension (Article 12).

6. In any kind of concrete structure the embedded steel has a tendency to displacement in its own longitudinal direction under load. The value of the steel as reinforcement depends upon its ability to withstand any forces tending to either push or pull it out; reinforced concrete is an impossibility without adhesion between steel and concrete, and destruction of the bond or adhesion means failure. The law governing adhesion is therefore the foundation of all theoretical study of reinforced concrete.

CHAPTER IV

ADHESION

7. The adhesion is measured in lbs. per square inch of embedded surface of the rod; its value is different for pulling and pushing tests. As the latter is somewhat higher it is sufficient to investigate the laws governing the pulling resistance and apply these laws to the pushing resistance also. The mathematical analysis of the bond stresses is impossible with the material on hand; even the test-data are meager and often contradictory. We know, however, that the following statements are approximately true, so that an embedded rod pulls out of the concrete block:

(*a*) when the stress in the steel reaches the elastic limit of the steel.

(*b*) when the tensile resistance of the concrete, in a lateral direction, is reached, because the block splits.

(*c*) when, instead of splitting, the concrete around the rod expands sufficiently to let the irregularities of the rod pass through.

(*d*) when the adhesion is destroyed.

Obviously, then, the designer must keep the *steel stress* well below the elastic limit, allowing for this and other reasons an ample factor of safety, while, at the same time, the *concrete* must be strong enough to meet the demands made upon it. Hence the diameter of the concrete block, or the thickness of the piece, is a very important factor, but unfortunately nothing is known in regard to the minimum allowable diameter, except that it is greater for deformed bars than for plain round or square rods. We can readily see that both the tensile strength and the coefficient of elasticity of the concrete has great influence upon the minimum allowable diameter; with a well-proportioned mortar and a mixture of say $1:2:3\frac{1}{2}$, we may perhaps suggest a diameter of concrete equal to ten diameters of the embedded steel as reasonably safe. In floor construction the bars usually find their ultimate anchorage in much larger bodies, the slab bars passing through the beams, the beam bars through the girders, and finally the girder bars through the columns. In all these cases the concrete is reinforced in a direction transverse to the direction of the pull, and the expansion in a lateral direction is thus partially or entirely prevented.

8. In the beam theory to be outlined below, great importance is attached to *the length of embedment beyond the supports* of the beam, in fact, this length represents the ultimate reserve of strength of the beam. It is usually considered good practice to imbed the bent bars from twenty-four to thirty-six diameters

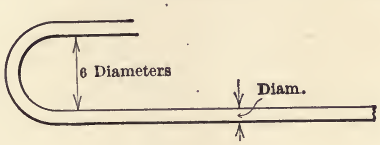


FIGURE 79.

beyond the support, using the lower figure for deformed bars stressed to 16,000 lbs./sq. inch, and the higher figure for deformed bars stressed to 20,000 lbs./ sq. inch. For plain bars, an additional hook is made on the end of the bar, equal in length to six diameters. In many cases the length of embedment here recommended cannot be obtained for the reason that there is no adjacent concrete into which the bars may be extended, as, for instance, in the case of a beam finding its bearing in an outside brick wall. The bars are then hooked, and the length of embedment calculated from the center of the seat to the end of the bar, including the curved end of the hook. Square hooks must be avoided, a gentle curve of, say, six times the radius of the bar is much more effective, and the more so the greater the radius of the bent (Figure 79).

9. The diameter referred to in the preceding paragraph is *not* the diameter of each individual rod or bar, unless the rods be spaced so far apart that each will pull out individually, leaving the concrete intact between. The diameter is that of a circle or other curved line in which all the rods may be enclosed if laid closely together. It follows that it is good practice to spread the rods out as much as possible; in a beam this is easily obtained by

bending some of the bars up over the support, as is also done for other important reasons. It is a common but inexcusable mistake to use a number of small diameter rods bunched together; —it is almost impossible to concrete such beams properly, and the fallacy of the argument leading to such construction should be evident.

CHAPTER V

COMPRESSION AND LATERAL EXPANSION

10. WITH few exceptions, materials submitted to deformation in one direction undergo deformations in all other directions. If the principal deformation is a shortening, the lateral deformation is a swelling, which must be taken as evidence of certain interior stresses in the body in a direction normal to that of the principal stress. These transverse stresses are of the greatest importance for materials like reinforced concrete, because, if not restrained, they bring about the premature failure of the concrete, while, if restrained, they may be used to increase the strength of the structure. Thus, as pointed out above, the transverse swelling affects the bond of an embedded rod; if restrained (by surrounding the bar with a coil of large diameter), the value of the bond may be increased as much as fifty per cent. or more. Even a loose stirrup circling the tension rod at the bottom of a beam increases the sliding resistance of the rod, so that a rod, covered at the most with two inches of concrete, may have the same sliding resistance as one embedded in a large body of concrete. Similarly, the Ransome Coil Coupling may be used with good results when splicing rods, although the rods should always be made in one continuous piece whenever it is practically possible. The coupling is made simply of a coiled piece of very heavy wire or a light bar surrounding the splice for its entire length, which should be equal to at least fifty diameters of the rods to be spliced (Figure 14).

11. In figure 80 a short block is shown loaded and compressed in one direction, thereby shortening the length of the vertical side from *aa* to *bb*. We notice now that the block expands in a horizontal direction, the diameter increasing from *cc* to *dd*. It requires very careful observation to discover this swelling in a concrete block, which usually fails along a diagonal such as *ae*, but, in any case, experiments with greased surfaces have shown

that when the friction is eliminated, the block fails along vertical planes such as *ff*.[1] It is therefore clear that longitudinal reinforcement in the direction of the compressive force is not very efficient, because the longitudinal rods simply add their own

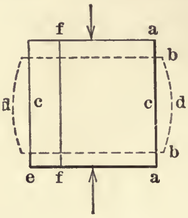


FIGURE 80.

strength to that of the concrete. The rods act as slender columns and have a tendency to buckle, so that if no other provisions are made, the strength of the rods is practically nil. To prevent buckling, horizontal ties or " hoops " are introduced, but it is evident that unless closely spaced the hoops are of little value. If therefore the column or block is to have vertical reinforcement, it must have *closely spaced horizontal hoops*, and these in turn prevent the concrete from breaking apart along the vertical planes *ff* described. In this way the hoops become a very efficient means of reinforcing.

12. In order to understand this fully, let us consider a cylinder filled with water, one end being equipped with a water-tight but frictionless piston. This piston will carry an immense weight on its upper surface; in fact, the entire system cannot fail before the water pressure within the cylinder exceeds the capacity of the cylinder walls, so that the cylinder bursts. The pressure within the cylinder is the same in all directions per unit of area; more particularly there is a horizontal (lateral) pressure on each and every square inch equal to the vertical pressure produced by the load on the piston. If now the cylinder is filled with sand instead of water, the conditions are only changed to this extent that the lateral pressure against the walls is *less* than before, so that it takes a greater load on the piston to burst the walls. Finally, if the cylinder is filled with liquid concrete, and the concrete is allowed to set hard, the pressure on the walls will be even

[1] According to tests by Foeppel and Mesnager. See, for instance, Considère, "Reinforced Concrete," page 120.

less than before, but the concrete will stand much higher pressure when enclosed in the cylinder than when free. This, then, is the principle of the "hooped column," that the horizontal metal jacket prevents the concrete from spreading and thereby increases its carrying capacity.[1]

For practical reasons it has been found impossible to use a continuous sheet of iron around the concrete; the horizontal reinforcement is always in the shape of hoops encircling the body of the concrete. Under pressure the concrete is sometimes seen to ooze out between the hoops, indicating the failure of the column, but usually the column fails by the bursting of the hoops or the complete disintegration of the concrete. In practical construction this need not concern us, as the stresses naturally always are low; more important is the relatively great shortening of the hooped column under working loads. This objection is overcome by the rational use of vertical rods, so that the true "hooped column" contains both hoops and verticals (Figure 81)

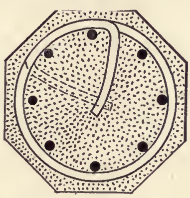


FIGURE 81.

13. The computation of a hooped column naturally centers around the calculation of the lateral pressure against the hoops. With a given concrete area F and a given load X the unit stress on the concrete becomes

$$\frac{X}{F} \text{ lbs./sq. inch} \begin{cases} \text{where } X \text{ is in lbs.} \\ \text{and } F \text{ in square inches.} \end{cases}$$

If we were dealing with water, the horizontal unit pressure would be the same. For concrete this is not the case; according to

[1] Attention is called to some very interesting tests by Prof. Ira H. Woolson, Eng. News, 1905, Nov. 2, — Steel tubes, 4″ in diameter and 12″ long, $\frac{1}{8}$″ thick walls, were filled with concrete. When seventeen days old, the tubes were tested in compression under loads as high as 120,000 to 150,000 lbs. The tubes bent out of shape, and shortened $3\frac{1}{2}$″, while the diameter increased from the original 4″ to 5″. When the tubes were removed, the concrete was found unbroken, solid, and perfect.

See also Trautwine, 1909, p. 1160.

experiment, the ratio between intensity of vertical stress and transverse stress is as 1 to 1/4.8. In other words, if the load produces a direct compressive stress of 480 lbs./sq. inch, the lateral pressure would at the same time be 100 lbs./sq. inch. It is now a simple matter to write up an expression giving the resistance due to the hoops, in a granular material having this same coefficient 4.8. Let us denote by u the ratio between this resistance and the volume of metal in the hoops, and let us denote by U a similar ratio obtained between the resistance due to vertical reinforcement, and the volume of the material in the verticals. The expressions u and U will then give the effect produced by a unit of material, used as hoops and as verticals. We find, assuming the same stress in hoops and verticals:

$$\frac{u}{U} = \frac{4.8}{2} = 2.4$$

which shows that pound for pound, the steel employed in the hoops is 2.4 times as effective as steel employed for longitudinal reinforcement.

14. The question is now to find the effect of the verticals. Assuming that they are well tied so as to prevent buckling of the individual rods, the unit stress on the verticals must be r times the stress on the concrete, if the sections are to remain plane as assumed in Article 4. It is easy to see that this assumption is on the safe side, because, if the sections curved, the stress in the steel might be very much more than r times that on the concrete, which latter forms the starting-point for our investigation. The value of r can only be indicated in a general way, as the properties of concrete vary greatly with the circumstances; let us assume $r = 20$. Then, if the unit stress on the concrete is 500 lbs./sq. inch, the stress on the steel becomes 10,000 lbs./sq. inch. Let F be the area of the concrete inside the hoops, and the allowable stress on this concrete C lbs./sq. inch. Let p denote the percentage of the verticals with reference to the volume of concrete, then the effective concrete area is $F \cdot \frac{100 - p}{100}$

and the area of the longitudinals $F \cdot \frac{p}{100}$;

hence the load carried by the concrete is $C \cdot F \cdot \frac{100 - p}{100}$ lbs.

Let the stress on the longitudinals be S lbs./sq. inch, then their share of the load becomes $S \cdot F \cdot \frac{p}{100}$ lbs.

Disregarding for the moment the influence of the hoops, the total carrying capacity of the reinforced column is

$$X_1 = C \cdot F \cdot \frac{100 - p}{100} + S \cdot F \cdot \frac{p}{100} \text{ lbs.} \tag{1}$$

while if allowance be made for the hoops, the percentage of which is q with reference to the concrete section, we have an additional strength due to the hoops equal to $2.4 \cdot S \cdot F \cdot \frac{q}{100}$

and the total carrying capacity of the column becomes

$$X_2 = C \cdot F \cdot \frac{100 - p}{100} + S \cdot F \cdot \frac{p}{100} + 2.4 \cdot S \cdot F \cdot \frac{q}{100} \text{ lbs.} \tag{2}$$

15. The formula (2) above is the true formula for a reinforced concrete column and should always be used except in localities where the building code prevents its use, in which case formula (1) may be used. In any case, hoops must be used, otherwise the column steel is of no value as reinforcement. For the hooped column, Considère, the inventor and first experimenter, recommends $p = q = 2$, which, with $C = 600$ lbs./sq. inch, and $r = 20$, gives

$$X_2 = 1400 \cdot F.$$

The hoops are spaced as closely as possible, leaving 1″ to 2″ clear space between the hoops to facilitate concreting. The spacing should under no circumstances exceed 1/6 of the diameter of the core. Finally the core is protected with a sufficient thickness of concrete to prevent rust and fire danger, about 1″ to 2″ of protection being required according to location and exposure.

The plain column has a vertical reinforcement varying from one to ten per cent. of the concrete area, although reinforcement in excess of say five per cent. should be avoided on account of the uncertainty of the strength of columns reinforced with large amounts of steel. It is evident that hoops are indispensable also in these columns; it is quite common to see the hoops spaced one or even two feet apart; such hoops are of no use. The steel cannot be depended upon to carry its load unless securely tied, say, 1/3 to 1/2 column diameters apart. With $p=4$, $S=12{,}000$, $C=600$, we have, for $r=20$:

$$X_1 = 1060 \cdot F.$$

16. Owing to difficulties in filling columns of small diameter, the diameter should not be much less than 10″ in any case, although there are many 8″ columns on record. On account of the danger of "column failure" the length should not exceed 15 diameters. It is possible to advance a theory for "long" columns, but experience shows that columns exceeding 15 diameters in length are rare indeed except in roof stories where the calculations often give very light sections. Moreover, all such theories depend alone upon theoretical considerations and have never been conclusively tested in the laboratory, so that in the rare cases where "long" columns are required it is better to make the column a little larger and avoid the uncertainties of the theory.

17. In tall buildings, or in warehouses, the column bars become quite heavy, and it is necessary to join the bars of the column above with those of the column below in a substantial manner. The most satisfactory way is to square the ends of the bars carefully and join them in rather closely fitting sleeves, taking care that each bar has a full bearing on the bar below. Absolute certainty is had by cutting threads on both bars and sleeves, and drawing the bars together tight with the sleeve, but this must be done with great care and under strict supervision in order to be at all effective; — unless carefully made this joint is worse than useless. When light bars are used they may be spliced by lapping by the required number of diameters, say about thirty, but this method is hardly to be recommended.

Each bar of the story above should find bearing on a bar below; the number of bars therefore increases downward in the building. The number of bars in each story should be such that the bars can be symmetrically arranged in the column, unless there is some extraordinary reason for arranging them otherwise (excentric loads). The proper arrangement of the column bars may sometimes cause the designer to spend a good deal of time in working out the correct solution, but he may feel assured that this time is well spent.

The hoops may be made from round or flat stock; the round stock may be obtained in long lengths and lends itself more readily to the requirements of the hooped column, especially where the reinforcement is manufactured in the shop, with permanent devices for coiling and fastening the hoops to the longitudinals. The hooped reinforcement may also be bought ready-made; quite frequently the manufacturer overlooks the importance of having the spiral hoops in one continuous piece from top to

FIGURE 82. COLUMN REINFORCEMENT
Loomis Building, Cleveland, Ohio. Alexis Saurbrey, Consulting Engineer

bottom, or, where the wire is joined, he makes a flimsy joint. It must be remembered that the hoops are tension-reinforcement and subject to all the rules governing the design of such bars. The best joint is made by simply bending the ends of the wire to the center of the column, making the loose end long enough to secure the requisite grip.

The hoops may also be made in individual pieces, slipped over the previously erected verticals and wired in place. If the hoops are neatly made an excellent job may be had in this way (Figure 82).

18. It follows from what is said above that a hooped column should preferably be made of a circular cross-section, because in that case the hoops are subject to direct tension only. In many cases the expense incidental to the use of circular forms is prohibitive; the concrete may then be made square or octagon in section while the circular form is retained for the hoops. In either case only the concrete *within the hoops* can be taken into account in the calculations. Sometimes the hoops are made square or rectangular, in which case they are less effective, but we do not know how much.

19. The top and bottom of each column deserves special attention as the tests made so far seem to indicate that these are the weakest parts of the column, although there are many exceptions to this rule. Suitable caps and bases are inexpensive, improve the appearance and increase the strength. Special investigation is always necessary at points where the concrete column finds a bearing on another material; the weight carried by the reinforcing rods must be distributed over such an area that the concrete in the column is not over-stressed. This is particularly true where the column rests on the footing; a steel base plate must be used to distribute the load on the rods, and the concrete must be enlarged so as to bring the average pressure within the allowable. This will be considered in detail under "footings."

20. Before leaving the subject of hooped columns, attention is called to the possibility of strengthening existing concrete columns with hoops wound around the outside of the column. In many cases it would be impossible to obtain satisfactory results in this manner, but when the concrete is of good quality, and the existing reinforcement is such as to give a sufficient amount of longitudinal reinforcement in the finished column, there should be no theoretical objections to this procedure. In practice it would of course be difficult to wrap the core tightly, but this is not absolutely necessary, as grout rich in cement may be forced between the hoops and the old concrete. Great care would be required in this operation, but it is not at all impossible, as has been shown by actual experiments on a small scale.[1]

[1] Considère: "Reinforced Concrete," page 175. The prism tested in this manner was allowed to set for three months, then wrapped with hoops and covered with cement, and tested after ten more days. The crushing

21. In many cases columns are subject to excentric loads, so that, in addition to the direct compressive force, a bending moment exists and must be taken care of. This will be considered in detail in Article 81.

strength was 10,500 lbs. per square inch. There were no longitudinals in this prism.

CHAPTER VI

BENDING

22. The theory of bending used for reinforced concrete beams is different from the ordinary "theory of flexure" as used for homogeneous beams in a few particulars only, and this difference is more apparent than real. We consider here only the point of maximum bending moment; this is also the point of maximum depth, and we may assume both the compressive and tensile resultant to be normal to a vertical section through this particular point, under the particular loading described below.

The notations used are as follows (Figures 83, 84):

d or D = depth from top of concrete to center of steel, inches.

xd = depth from top of concrete to neutral axis, inches.

$x = \frac{xd}{d}$ = ratio between the two preceding items.

d_1 = distance center of compression to center of tension, inches.

E_c and E_s = coefficients of elasticity for concrete and steel.

$r = \frac{E_s}{E_c}$ = ratio between these coefficients.

t or T = thickness of a flange, inches.

b = width of flange considered, inches.

$B = \frac{b}{12}$ = width of flange considered, feet.

n = thickness of stem of beam, inches.

r_c and r_s = deformations of concrete and steel, at extreme fiber.

C = unit stress on concrete in outside fiber, compression, lbs. per square inch.

S = unit stress in steel, tension, lbs. per square inch.

a = area of steel, square inches.

s_t = total pull in steel in tons.

c_1, c_2, c_3 = coefficients relating to balanced design of the section.

α, β = coefficients relating to T-beams with greater than minimum depth.

w = dead plus live load on slab, lbs. per square foot.
l = span in feet.
q = factor of continuity.
M = bending moment in tons-inches.
$m = 2000\,M$ = bending moment in lbs.-inches.

23. In regard to the load, we will let all loads act in the same vertical plane along the center line of the beam as is usually the case in practical construction. This excludes at once all loads which would cause the beam to rotate around its longitudinal axis and all loads which would cause the beam to slide in its own direction.

24. In regard to the deformations, we will consider these as very small in comparison with the dimensions of the beam, so that the stresses are considered as acting upon the original cross-sections, not upon the deformed cross-sections or upon the deflected beam.

25. This does not mean that the change of shape of the section is of no importance. In figure 83 a vertical section is shown

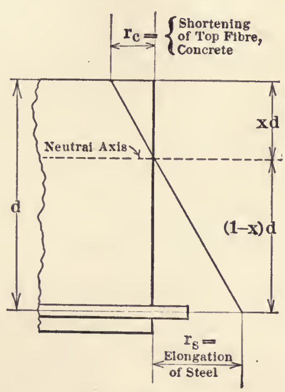


FIGURE 83.

with the deformations produced by the bending of the beam; we assume sections plane before bending to remain plane after bending. Inspection of the diagram shows that the upper fibers are shortened, the lower fibers extended under the load; — the neutral axis forms the division line between shortened and extended fibers. The assumption of plane sections is evidently equivalent to assuming that the deformation of any fiber is in direct proportion to its distance from the neutral axis, and thus we get the equation:

$$\frac{r_c}{r_s} = \frac{x\,d}{(1 - x)\,d} = \frac{x}{1 - x} \tag{3}$$

26. We further assume that the stress on any small unit is directly proportional with the deformation; this gives the equations:

$$\left.\begin{array}{lll} \text{for concrete} & C = r_c E_c \text{ or } r_c = \dfrac{C}{E_c} \\ \text{for steel} & S = r_s E_s \text{ or } r_s = \dfrac{S}{E_s} \end{array}\right\} \frac{r_c}{r_s} = \frac{C}{S} \cdot \frac{E_s}{E_c} \tag{4}$$

27. We shall later[1] have occasion to use the moment of inertia of the section. It is therefore necessary to note that the assumptions made in the preceding paragraphs are identically the same as those used in the "common theory of flexure" which leads to the well-known expression

$$\sigma = \frac{M}{I} \cdot e \tag{5}$$

where σ = stress per unit
M = bending moment
I = moment of inertia
e = distance from neutral axis to fiber considered.

The new feature in a reinforced concrete beam is now that in writing up the moment of inertia we have to disregard the concrete below the neutral axis entirely, and instead consider the steel area. To this we shall return later.

28. Combining now equations 3 and 4 we find

$$\frac{C}{S} \cdot \frac{E_s}{E_c} = \frac{x}{1 - x}$$

hence

$$x = \frac{1}{1 + \dfrac{S}{Cr}} \tag{5a}$$

which expression determines the location of the neutral axis.

29. If now a vertical section is laid across the beam and stresses added on and in the section to represent the removed portion of the beam, the beam will remain in equilibrium. Let us project all forces and stresses on a horizontal line: then the

[1] Article 79.

loads, being vertical, give no projections, and similarly the stresses acting in the vertical section itself disappear. There remain only the normal stresses acting against the section; as equilibrium presupposes that the sum of all the projected forces and stresses is zero, we have

$$\left.\begin{array}{c}\text{horizontal component}\\ \text{of stresses}\\ \text{on tension side}\end{array}\right\} = \left\{\begin{array}{l}\text{horizontal component}\\ \quad\text{of stresses}\\ \text{on compression side.}\end{array}\right.$$

Referring now to Figure 84, the area stressed in compression is xd inches high, b inches wide, and the average stress $\frac{1}{2}C$ lbs. per square inch. Hence

$$\text{total compression} = \tfrac{1}{2}C \cdot xd \cdot b \text{ lbs.}$$

Denoting by s_t the total pull in the steel in tons, we have, neglecting the tension in the concrete,

$$\text{total tension} = s_t \cdot 2000 \text{ lbs.}$$

$$\text{Hence} \quad s_t \cdot 2000 = \tfrac{1}{2}Cxdb$$

which gives $$s_t = \frac{Cxdb}{4000} \quad \text{or} \quad s_t = \frac{c_2}{12}bd \text{ tons}$$

where $$c_2 = \frac{Cx}{333} \tag{6}$$

30. Two more conditions must be fulfilled in order to create equilibrium: (1) the sum of all stresses and forces must be zero when projected upon a vertical line (when the loads are vertical, Article 23); this condition we will consider later under " U-bars." (2) The sum of all moments around any arbitrary point must be zero. Select for this point the point of application of the compressive stresses; the moment of the *loads* is then the " bending moment " m inch-lbs. The moment of the *stresses* is 2000 $s_t\, d_1$ inch-lbs. We must then have

$$0 = m - 2000\, s_t \,.\, d_1$$

but according to the diagram (Figure 84)

$$d_1 = (1 - \tfrac{1}{3}x)\, d$$

hence $$0 = m - 2000 \cdot s_t \cdot (1 - \tfrac{1}{3}x)\, d.$$

Eliminating s_t we find

$$m = \tfrac{1}{2}Cxb\,(1 - \tfrac{1}{3}x)\, d^2$$

hence $d = \dfrac{1}{c_1}\sqrt{\dfrac{m}{b}}$ inches where $$c_1 = \sqrt{\tfrac{1}{2}Cx\,(1 - \tfrac{1}{3}x)} \tag{7}$$

Finally the steel area: $a = \dfrac{2000}{S} s_t$ square inches.

31. The formulas apply to all rectangular beams and therefore also to slabs. As we disregard the tensile resistance of the concrete, the concrete below the neutral axis does not in any way enter into the calculations at this point, and the formulas are therefore also correct for T-beams where the bottom of the flange coincides with the neutral axis. In this case the thickness of flange simply becomes

$$t = xd \text{ inches.}$$

32. We have now everything required to proceed with the design:

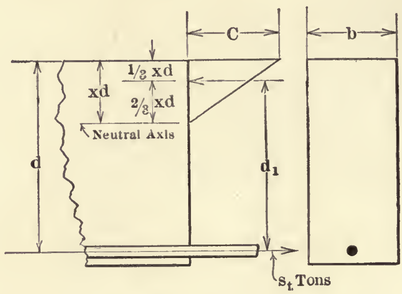


FIGURE 84.

The depth in inches: $$d = \frac{1}{c_1}\sqrt{\frac{m}{b}} \qquad (8)$$

The pull in the steel, tons: $$s_t = \frac{1}{12} c_2 bd \qquad (9)$$

The thickness of flange, inches: $$t = xd \qquad (10)$$

The steel area, square inches: $$a = \frac{2000}{S} s_t \qquad (11)$$

Simple as these formulas are they can only be used when the values of the coefficients x, c_1 and c_2 are known, and these values in turn depend upon the allowable stresses and the factor r. The Tables I, II, and III give full information in regard to the values of the coefficients; it will be noticed that the same tables may be used for any value of r, by simply shifting the position of the S-column in relation to the values of the coefficients. On the left the ordinarily used S-column is indicated, corresponding to $r=15$; while on the right, the S-columns corresponding to $r=12$ and $r=20$ are shown. Usually existing building codes and engineers' specifications call for $r=15$ in bending-problems, but this selection is arbitrary, and other values of r may very well be used. It is impossible to predict the coefficient of elasticity of concrete beforehand, and even if determined by careful experiment there is no reason to believe that it would remain the same on the building to be erected as in the laboratory, while it is quite certain that it changes materially from day to day as temperature and moisture affect the mixture used for the concrete.

In Table IV values of the coefficient $c_3 = 1 - \frac{1}{3}x$ are indicated; the use of this table will be clear from the analysis above. In Table V the percentage of steel in a rectangular beam is indicated corresponding to $r=15$; when the allowable stresses are decided upon, the percentage of steel in the section is a fixed quantity.

33. In the formulas above all dimensions are in inches, the moment in inch-lbs., the pull s_t in tons. In practical design it is usually convenient to have the bending moment in inch-tons, M, and the width in feet, B. The formulas then become:

The depth in inches: $$d = \frac{12.9}{c_1} \cdot \sqrt{\frac{M}{B}} \qquad (8a)$$

The pull in the steel, tons: $$s_t = c_2 B d \qquad (9a)$$

The thickness of flange, inches: $$t = xd \qquad (10a)$$

The steel area, square inches: $$a = \frac{2000}{S} s_t \qquad (11)$$

These formulas are different from those given above in this respect only, that the figures handled are much smaller and therefore it becomes easier to avoid mistakes, as figures of two or three places may be multiplied and divided, etc., approximately, without the use of paper and pencil, so that all calculations are easily verified.

34. The formulas given above apply, as stated, to slabs, to rectangular beams, and to T-beams in which the neutral axis coincides with the bottom line of the flange. Usually these two lines do not coincide, so that it becomes necessary to make further investigation in order to derive a general formula. The formulas given above have this peculiarity, that, for a given width of beam, the dimensions derived are *minimum* dimensions which cannot be decreased without adding to the stress on the material, thus exceeding the allowable stresses on which the design was based. Briefly stated, the problem before us consists in finding

TABLE I. Depth of Neutral Axis $= xd$

$$x = \frac{1}{1 + \frac{S}{Cr}}$$

$r = 15$	Table I x								$r = 12$	$r = 20$
S = 24,000	.158	.200	.238	.272	.304	.333	.360	.385	S = 19,200	—
22,000	.170	.214	.254	.290	.322	.352	.380	.405	17,600	—
20,000	.184	.231	.272	.310	.344	.375	.404	.429	16,000	—
18,000	.200	.250	.294	.333	.369	.400	.429	.454	14,400	—
16,000	.219	.272	.318	.360	.397	.429	.458	.483	12,800	S = 24,000
14,000	.244	.300	.349	.392	.429	.463	.491	.519	11,200	21,300
12,000	.272	.333	.385	.429	.468	.500	.529	.556	9,600	18,600
10,000	.310	.376	.429	.474	.513	.546	.574	.602	—	16,000
C =	300	400	500	600	700	800	900	1000		
$r = 15$									$r = 12$	$r = 20$

TABLE II. Effective Depth

$$d = \frac{12.9}{c_1}\sqrt{\frac{M}{B}} \text{ or } d = \frac{1}{c_1}\cdot\sqrt{\frac{m}{b}}$$

$r = 15$	Table II c_1								$r = 12$	$r = 20$
S = 24,000	4.7	6.1	7.4	8.6	9.8	11.0	12.0	13.0	S = 19,200	—
22,000	4.9	6.3	7.6	8.8	10.0	11.2	12.2	13.2	17,600	—
20,000	5.1	6.5	7.9	9.1	10.3	11.5	12.5	13.5	16,000	—
18,000	5.3	6.8	8.1	9.4	10.6	11.8	12.8	13.9	14,400	—
16,000	5.5	7.0	8.4	9.7	11.0	12.1	13.2	14.2	12,800	S = 24,000
14,000	5.8	7.3	8.8	10.1	11.3	12.5	13.6	14.7	11,200	21,300
12,000	6.1	7.7	9.2	10.5	11.7	12.9	14.0	15.1	9,600	18,600
10,000	6.5	8.2	9.6	11.0	12.2	13.4	14.5	15.5	—	16,000
C =	300	400	500	600	700	800	900	1000	—	—
$r = 15$									$r = 12$	$r = 20$

TABLE III. Total Pull in Steel

$$s_t = c_2\, Bd \text{ or } s_t = \frac{1}{12}\, c_2\, bd$$

$r = 15$	Table III c_2								$r = 12$	$r = 20$
S = 24,000	.14	.24	.36	.49	.64	.80	.97	1.16	S = 19,200	—
22,000	.15	.26	.38	.52	.68	.85	1.03	1.22	17,600	—
20,000	.17	.28	.41	.56	.72	.90	1.10	1.29	16,000	—
18,000	.18	.30	.44	.60	.78	.96	1.16	1.36	14,400	—
16,000	.20	.33	.48	.65	.83	1.03	1.24	1.45	12,800	S = 24,000
14,000	.22	.36	.52	.71	.90	1.12	1.33	1.56	11,200	21,300
12 000	.25	.40	.58	.77	.99	1.20	1.43	1.67	9,600	18,600
10,000	.28	.45	.64	.86	1.08	1.31	1.55	1.80	—	16,000
C =	300	400	500	600	700	800	900	1000	—	—
$r = 15$									$r = 12$	$r = 20$

TABLE IV. ARM OF "COUPLE OF STRESSES."

$$s_t = \frac{M}{d_1};\ d_1 = c_3 d;\ c_3 = 1 - \frac{1}{3}x$$

$r = 15$	TABLE IV $c_3 = 1 - \frac{1}{3}x$								$r = 12$	$r = 20$
S = 24,000	.95	.93	.92	.91	.90	.89	.88	.87	S = 19,200	—
22,000	.94	.93	.92	.90	.89	.88	.87	.87	17,600	—
20,000	.94	.92	.91	.90	.89	.88	.87	.86	16,000	—
18,000	.93	.92	.90	.89	.88	.87	.86	.85	14,400	—
16,000	.93	.91	.89	.88	.87	.86	.85	.84	12,800	S = 24,000
14,000	.92	.90	.88	.87	.86	.85	.84	.83	11,200	21,300
12,000	.91	.89	.87	.86	.84	.83	.82	.82	9,600	18,600
10,000	.90	.88	.86	.84	.83	.82	.81	.80	—	16,000
C =	300	400	500	600	700	800	900	1000	—	—
$r = 15$									$r = 12$	$r = 20$

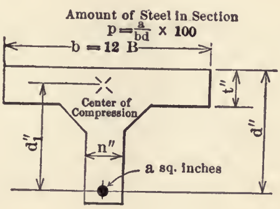


TABLE V

$r = 15$	p							
S = 24,000	.098	.167	.247	.339	.442	.553	.672	.801
22,000	.115	.196	.288	.393	.510	.636	.771	.920
20,000	.138	.231	.339	.465	.602	.750	.907	1.07
18,000	.166	.278	.406	.553	.714	.884	1.07	1.26
16,000	.204	.339	.493	.672	.865	1.07	1.27	1.50
14,000	.261	.428	.621	.839	1.07	1.33	1.58	1.85
12,000	.339	.554	.799	1.07	1.36	1.66	1.98	2.31
10,000	.463	.750	1.07	1.42	1.79	2.17	2.57	2.99
C =	300	400	500	600	700	800	900	1000

the effect on the T-beam of an increase in depth, which must, in order to balance the design, be accompanied by a corresponding decrease in thickness of flange and amount of steel.

35. Let, then, Figure 85*a* represent a section of T-shape of minimum dimensions, having a depth d, a thickness of flange t_a, and a total pull in the steel of s_a tons. Let, further, Figure 85*b* represent a new section with a new, larger depth $D = ad$.

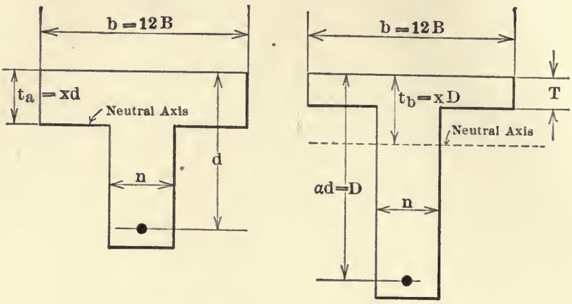


FIGURE 85*a*. FIGURE 85*b*.

The given M and B remain the same; we wish to determine the new values s_b and T pertaining to Figure 85*b*.

We observe, then, that the proportionate depth x of the neutral axis is the same in the two beams, because the allowable stresses are the same, so that the depth of neutral axis is calculated as $t_a = xd$ in the first beam and as $t_b = xD$ in the second. The "effective depth" in the first beam is

$$d_1 = (1 - \tfrac{1}{3}x)\,d$$

and in the second approximately

$$D_1 = (1 - \tfrac{1}{3}x)\,D = (1 - \tfrac{1}{3}x)\,ad$$

The approximation consists in disregarding the tendency of the center of compression to rise on account of the removal of the concrete near the neutral axis; the discrepancy is negligible in most cases and on the safe side. Since now

$$s_a = \frac{M}{d_1} \quad \text{and} \quad s_b = \frac{M}{D_1} = \frac{M}{ad_1}$$

we get the equation $s_a = as_b$ where $s_a = \dfrac{C}{4000} \cdot t_a \cdot b$

and, by reference to Figure 86

$$s_b = \frac{C + C_T}{4000} \cdot T \cdot b + \frac{C_T}{4000} \cdot (at - T) \cdot n.$$

Introducing these values in $s_a = as_b$ we get after some reduction an equation

$$t_a = \alpha\left(2 - \frac{T}{\alpha t_a}\right)T + \alpha^2\left(1 - \frac{T}{\alpha t_a}\right)^2 \cdot t_a \cdot \frac{n}{b}$$

Solving for $\left(\frac{T}{t_a}\right)$ and denoting by β the value of this ratio we find

$$\beta = \alpha - \sqrt{\frac{\alpha^2 - 1}{1 - \frac{n}{b}}} \cdot \qquad (12)$$

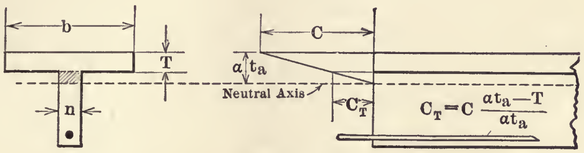


FIGURE 86.
Showing the stresses in the beam of Figure 85*b*.

The thickness of flange in our new beam is now

$$T = \beta t_a = \beta x d = \frac{\beta}{\alpha} \cdot x \cdot D$$

and the new total pull in the steel is

$$s_b = \frac{s_a}{\alpha} = \frac{c_2}{\alpha} \cdot B \cdot d = \frac{c_2}{\alpha^2} \cdot B \cdot D$$

corresponding to the new depth $D = \alpha d$.

36. We have now the following general formulas for any reinforced concrete T-section:

The depth, in inches	$D = \alpha \,.\, \frac{12.9}{c_1} \cdot \sqrt{\frac{M}{B}}$	(13)
The total pull in the steel, in tons	$s_t = \frac{c_2}{\alpha^2} \cdot B \cdot D$	(14)
The thickness of flange, in inches	$T = \frac{\beta}{\alpha} \cdot x \cdot D$	(15)
The steel area	$a = \frac{2000}{S} \cdot s_t$	(11)

where M is the bending moment in tons-inches, α an arbitrary coefficient larger than unit, while the width B feet may be given or selected. The coefficient β is derived from α by formula 12 but to facilitate calculations, Table VI has been prepared giving the values of β for various combinations of α and n/b. This latter ratio has little influence on the result within the ordinary limits, and Table IX may also be used in cases where n/b is different from 1/4, if the variation is not too large, although prepared especially for n/b = 1/4.

37. The theory of T-beams is of great importance as all the floor systems in common use involve this principle. Lately, beamless floors have come into use, and to these we shall return later; the beam and slab floors may be divided into two groups, the first including solid concrete floors, the second what is known as "tile-concrete" floors. The first of these two is by far the oldest, but the "tile-concrete" is gaining in favor with every day, and justly so, as its cost is less for light buildings owing primarily to the simplicity of the form work. The long flat ceilings are well adapted to modern store building and office-structures, especially where the loads are light and distributed. These floors have a flat portion supported on main girders *A* (Figure 87), the flat portion consisting of ribs *B* built between

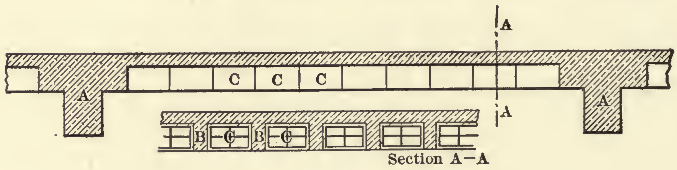


FIGURE 87.

rows of hollow tiles *C* and a top covering of two or more inches of concrete, thus forming series of comparatively light T-beams side by side. The main girders are also of T-shape, the flanges being formed by leaving out the requisite number of tiles next to the stem of the girder. Sometimes lighter tiles *D* are used near the stem, in which case the flange becomes thinner than when the tiles are omitted entirely, Figure 88. The commercial sizes

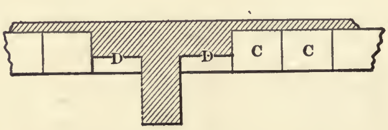


FIGURE 88.

of tiles are usually 12″ x 12″ in plan, the depth ranging from 4″ to 12″ or even 16″. When designing, it becomes necessary to proportion the depth of floor so as to allow for these commercial sizes; the function of the tiles is simply to create a void in the concrete, and they do not enter into the calculated strength of the floor. The calculations require considerable time if exact, and tables VII and VIII have therefore been prepared for $C=700$ and $S=20{,}000$ and 16,000 respectively. These tables show at a glance the depth of tile and thickness of concrete required for any given bending moment, together with the corresponding pull in the steel. Note, however, that the bending moment must be calculated for a width b of slab equal to the distance between centers of ribs. If other allowable stresses are assumed than those for which the tables have been prepared, we may easily prepare new tables. We have in all the preceding formulas, that the bending moment is directly proportional to the square of the coefficient c_1, while the total pull in the steel is directly proportional to the coefficient c_2. But we have

$$\frac{c_1^2}{c_2} = \text{a coefficient times } c_3$$

A glance at Table IV shows that c_3 itself is practically a constant within fairly wide limits, so that, for the allowable stresses in ordinary use, we may make

$$\frac{c_1^2}{c_2} = \text{constant.}$$

It follows that the new tables are prepared from the tables here given by multiplying both the bending moment and the pull in the steel of the old table with a factor; this factor is the same for both items and is

$$\frac{\text{the new value of } c_2}{\text{the old value of } c_2}$$

A completed floor of this kind is shown in Figure 89.

38. **Flat Slabs.** If, in formulas $8a$ and $9a$, we make $B=1$, we have the slab formulas

$$d = \frac{12.9}{c_2}\sqrt{M} \quad \text{and} \quad s_t = c_2 d$$

But the load on the slab is usually given and in lbs./sq. foot; denoting by w the total dead and live load in lbs./sq. foot, the bending moment per foot width becomes

$$M = \frac{1}{q} \cdot \frac{w}{2000} \cdot l^2 \cdot 12 \text{ tons-inches}$$

TABLE VI. β

Increasing the Depth from d to $D = ad$

$$D'' = a\frac{12.9}{c_1}\cdot\sqrt{\frac{M}{B}};\quad s_t = \frac{c_2}{a^2}\cdot B\cdot D;\quad T'' = \frac{\beta}{a}\,x\,D.\quad \beta = a - \sqrt{\frac{a^2-1}{1-\frac{n}{b}}}$$

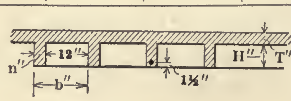
See Table IX for special case $\frac{n}{b} = \frac{1}{4}$

	β								
a = 1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
1.1	.64	.62	.59	.56	.50	.44	.38	.25	.08
1.2	.54	.50	.46	.41	.34	.26	.15	—	—
1.3	.47	.42	.37	.31	.23	.12	—	—	—
1.4	.42	.37	.30	.23	.14	.01	—	—	—
1.5	.38	.32	.25	.16	—	—	—	—	—
1.6	.35	.28	.21	.11	—	—	—	—	—
1.7	.32	.25	.16	.06	—	—	—	—	—
1.8	.30	.22	.13	.01	—	—	—	—	—
1.9	.28	.20	.09	—	—	—	—	—	—
2.0	.27	.18	.06	—	—	—	—	—	—
n/b =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8

Tables of $\frac{M}{s_t} = \frac{\text{inch-tons}}{\text{tons}}$; $r = 15$

TABLE VII $S = 20{,}000$ $C = 700$

T'' = 7.0	—	—	—	—	—	—	393/20.6
6.5	—	—	—	—	—	—	372/20.0
6.0	—	—	—	—	—	—	350/19.3
5.5	—	—	—	—	—	276/17.1	329/18.6
5.0	—	—	—	—	204/14.9	255/16.5	308/17.8
4.5	—	—	—	—	189/14.3	236/15.7	284/16.9
4.0	—	62/8.3	93/10.1	132/12.1	174/13.6	216/14.8	259/15.9
3.5	—	55/7.7	85/ 9.6	120/11.4	158/12.7	196/13.8	235/14.8
3.0	26/5.3	48/7.2	76/ 9.1	108/10.6	141/11.7	175/12.2	210/13.6
2.5	21/4.8	42/6.8	67/ 8.4	94/ 9.7	123/10.6	153/11.5	184/12.3
2.0	17/4.4	37/6.2	57/ 7.6	80/ 8.6	104/ 9.4	130/10.2	158/10.9
1.5	14/3.9	29/5.4	46/ 6.5	65/ 7.3	85/ 8.0	107/ 8.7	132/ 9.3
1.0	10/3.2	21/4.4	35/ 5.0	50/ 5.7	66/ 6.4	84/ 6.9	105/ 7.5
0.5	6/2.3	13/3.0	23/ 3.5	33/ 4.0	46/ 4.5	60/ 5.0	76/ 5.5
0.0	1/0.6	4/1.1	9/ 1.6	15/ 2.0	23/ 2.5	33/ 3.0	45/ 3.5
H'' =	4	6	8	10	12	14	16



$$M = \frac{1}{q} \cdot \frac{w}{2000} \cdot l^2.b$$

TABLE VIII $S = 16{,}000$ $C = 700$

$T'' = 7.0$	—	—	—	—	—	—	443/23.9
6.5	—	—	—	—	—	—	420/23.0
6.0	—	—	—	—	—	—	396/22.2
5.5	—	—	—	—	—	312/19.7	372/21.4
5.0	—	—	—	—	231/17.1	288/19.0	348/20.5
4.5	—	—	—	—	214/16.4	266/18.0	320/19.4
4.0	—	70/9.5	105/11.6	149/13.9	197/15.6	244/17.0	292/18.3
3.5	—	62/8.9	98/11.0	136/13.1	178/14.6	221/15.9	266/17.0
3.0	29/6.1	55/8.3	86/10.5	122/12.2	159/13.5	198/14.6	238/15.7
2.5	24/5.5	47/7.8	75/ 9.7	106/11.1	138/12.2	173/13.2	208/14.2
2.0	19/5.1	41/7.1	64/ 8.7	90/ 9.9	118/10.8	147/11.7	179/12.5
1.5	16/4.5	33/6.2	52/ 7.5	73/ 8.4	98/ 9.2	121/10.0	149/10.7
1.0	11/3.7	24/5.1	40/ 5.8	56/ 6.6	74/ 7.4	94/ 7.9	119/ 8.6
0.5	7/2.6	15/3.5	26/ 4.0	38/ 4.6	52/ 5.2	68/ 5.8	86/ 6.3
0.0	1/0.7	5/1.3	10/ 1.8	17/ 2.3	26/ 2.9	38/ 3.5	51/ 4.0
$H'' =$	4	6	8	10	12	14	16

which gives the very convenient formulas (16)

$$d = \frac{l}{c_1} \cdot \sqrt{\frac{w}{q}} \text{ inches}$$

and $$s_t = c_2 d \text{ tons.} \tag{17}$$

Here the span l is expressed in feet, and the factor q is equal to 8 for non-continuous construction, and from 10 to 16 for continuous construction.

39. If reinforced in both directions, and supported on all four sides, the slab is calculated by the formulas above, dividing the total load w into two portions w_1 and w_2 where $w_1 + w_2 = w$. If we denote by L_1 and L_2 the span in each direction, we have the arbitrary formulas for the division of the load:

$$w_1 = w \cdot \frac{L_1^4}{L_1^4 + L_2^4}$$

and

$$w_2 = w \cdot \frac{L_2^4}{L_1^4 + L_2^4}$$

The heaviest of these is now assigned to the shortest span, and determines the depth and the reinforcement running the short way, the cross reinforcement is designed in a similar manner using the other load. In case of square panels the two loads become equal, each one-half of the total.

The formula is entirely irrational and the only reason it is

TABLE IX $\frac{n}{b} = \frac{1}{4}$

$$\beta = \alpha - \sqrt{\tfrac{4}{3}(\alpha^2 - 1)}$$

α^2	α	β	β/α
1.00	1.000	1.000	1.00
1.01	1.005	0.890	0.89
1.02	1.099	0.847	0.84
1.03	1.015	0.815	0.80
1.04	1.020	0.789	0.77
1.05	1.025	0.767	0.75
1.06	1.030	0.747	0.73
1.08	1.039	0.712	0.69
1.10	1.049	0.684	0.65
1.12	1.058	0.658	0.62
1.14	1.068	0.636	0.60
1.16	1.077	0.616	0.57
1.18	1.086	0.596	0.55
1.20	1.096	0.579	0.53
1.25	1.118	0.541	0.49
1.30	1.140	0.508	0.44
1.35	1.162	0.479	0.41
1.40	1.183	0.452	0.38
1.45	1.204	0.430	0.36
1.50	1.225	0.409	0.33
1.60	1.265	0.371	0.29
1.70	1.304	0.338	0.26
1.80	1.342	0.308	0.23
1.90	1.378	0.283	0.21
2.00	1.414	0.258	0.18
2.25	1.500	0.209	0.14
2.50	1.581	0.167	0.11
2.75	1.658	0.130	0.078
3.00	1.732	0.099	0.058
3.50	1.870	0.045	0.024
4.00	2.000	0.000	0.000
α^2	α	β	β/α

given here is that it is on the safe side and better than other existing formulas.

The supporting girders are designed with reference to the load brought upon them by the particular direction which they support, and the bending moment is often increased above the calculated because the load seems rather more concentrated towards the center.

FIGURE 89. TILE CONCRETE CONSTRUCTION, READY FOR PLASTERER

Wise Building, Cleveland, Ohio. Alexis Saurbrey, Consulting Engineer

Discussion of Tables I to IX.

40. TABLE I.

$$x = \frac{1}{1 + \frac{S}{rC}}$$

The value of x determines the location of the neutral axis, xd being the distance from compression face to neutral axis.

We have seen that the position of the neutral axis within the section of a T-shaped beam leads to the division of T-beams into two groups, according to whether the neutral axis falls above or below the bottom line of the flange. In this latter case we introduce the coefficients α and β, and the problems contain an arbitrary element which is absent in beams of the first type, where the dimensions depend mutually upon one another as in formula (8). The table shows that the neutral axis only in exceptional cases approaches the middle of the beam where it is located in all symmetrical beams following Hooke's Law (steel for example). It is obvious that a greater amount of steel is required for low steel stresses than for high; we therefore see that the neutral axis is lowered by increasing the amount of steel.

41. Table II. $c_1 = \sqrt{\frac{1}{2} C x \left(1 - \frac{1}{3} x\right)}$

We have $d = \frac{12.9}{c_1} \cdot \sqrt{\frac{M}{B}}$

The smallest possible value of d is obtained when a large value of c_1 is used, or, in other words, when *high* concrete stresses are combined with *low* steel stresses. The influence of the concrete stresses is much more pronounced than that of the steel stresses; it is, therefore, not economy to increase the amount of steel in order to save on the concrete. It is not impossible to analyze this problem mathematically, but owing to variation in unit prices it seems hardly worth while. The possibility of decreasing the depth of construction by using high concrete stresses and low steel stresses may, however, be of importance in special cases where the head room is limited.

42. Table III. $c_2 = \frac{Cx}{333}$

The total pull in the steel is $s_t = c_2 \cdot \frac{b}{12} \cdot d.$

The total amount of steel is $a = s_t \cdot \frac{2000}{S}$

so that $a = \frac{2000}{S} \cdot c_2 \cdot \frac{bd}{12}$ or $c_2 = \left(\frac{a}{bd}\right)\left(\frac{S}{166}\right)$

The coefficient c_2, then, is a measure of the amount of steel used for a given cross-section, bd being the area of the cross-section in square inches. We note that

$$\frac{a}{bd} \text{ times } 100$$

is the percentage of steel required for the beam; if we denote the percentage by p we have

$$p = c_2 \cdot \frac{16600}{S}$$

This expression has been used in calculating Table V.

43. Table IV. $c_3 = 1 - \frac{1}{3}x$

In general the total pull in the steel is obtained by dividing the bending moment by a certain lever arm d_1, equal in length to the distance between the centers of compression and tension. Reference to Figure 84 gives at once

$$d_1 = \tfrac{2}{3}\,xd + (1 - x)\,d = (1 - \tfrac{1}{3}\,x)\,d = c_3 d$$

When d is known we get d_1 as c_3d; Table IV gives the values of c_3 for various combinations of stresses.

44. Table V. $p = \frac{a}{bd} \times 100 = c_2 \frac{16600}{S}$

The percentage of steel has but little interest for the practical designer as the problems usually present themselves. The table is added for the convenience of those who are in the habit of selecting the percentage of steel rather than determine the allowable stresses. The table is correct only for such beams where $\alpha = 1$ and $r = 15$.

45. Table VI will be found useful when designing T-beams of larger than minimum depth. When we have selected α as explained in connection with formula 12, etc., the corresponding β is found by Table VI for any value of n/b. The method of design will be clearly evident from the example in Article 47. Table IX is a more extensive table for the special case where $n/b = \frac{1}{4}$, which is a common value in practice. The variation of n/b does not affect the values very much, so that for small values of α Table IX may be used for other values of n/b than $\frac{1}{4}$.

46. Tables VII and VIII. Tile-concrete floors.

The use of these tables is best explained by an example. The span of the flat portion is 20 feet; the total dead and live load is assumed to be 250 lbs. per square foot. With 4″ ribs we get the width of beam

$$b = 4 + 12 = 16''$$

and the corresponding bending moment in inch-tons

$$M = \tfrac{1}{8} \times \frac{250}{2000} \times 20^2 \times 16 = 100 \text{ inch-tons.}$$

If the allowable stresses are $S = 20{,}000$ and $C = 700$, we must use Table VII, and we see at once that we can use either a 10″ tile with about $2\frac{3}{4}''$ concrete, or a 12″ tile with 2″ concrete. As we do not wish to have less than 2″ of concrete over the tiles, we cannot use the larger tiles economically. If we select 12″ tiles and 2″ concrete, the corresponding pull in the steel is 9.4 tons according to the table, requiring .94 square inches of steel, for instance, one $\frac{3}{4}''$ square and one $\frac{5}{8}''$ square bar.

It should not cause surprise that the moments tabulated in Table VIII are larger than the corresponding values of Table VII, although the allowable stress on the steel is smallest in Table VIII. The explanation is given in the remarks under Table II, Article 41, and, in accordance with the statements made there, it will be seen that the larger moments of Table VIII are obtained only by increasing the steel areas.

Table IX. See Table VI, Article 45.

47. Example 1. **T-Sections.** Continuing the example given above under the discussion of Tables VII and VIII, we proceed as follows to design the girder:

The load on the floor is 250 lbs. per square foot, the span of the flat portion on each side of the girder is 20′ – 0″, and the girder therefore carries a load of $250 \times 20 = 5{,}000$ lbs. per lineal foot, to which should be added the weight of the girder itself. Assuming this item to be included in the 5,000 lbs., and assuming a span of 24′ – 0″ for the girder, the bending moment on the girder becomes

$$M = \tfrac{1}{8} \times \frac{5000}{2000} \times 24^2 \times 12 = 2160 \text{ inch-tons.}$$

We decide to use high tension steel, for which $S = 20{,}000$, and we allow $C = 700$ lbs. per square inch on the concrete. We get then from Table II: $c_1 = 10.3$; from Table III: $c_2 = .72$; and from Table I: $x = .344$, and we may now proceed with the design, using formulas (8a), (9a), (10a), and (11). The width of flange,

B, may be selected arbitrarily. Let us make $B = 4' - 0''$. Then, by

$$\text{(8a)} \quad \ldots\ldots\ldots d = \frac{12.9}{c_1} \cdot \sqrt{\frac{M}{B}} = \frac{12.9}{10.3} \cdot \sqrt{\frac{2160}{4}} = 29.1''$$

$$\text{(9a)} \quad \ldots\ldots\ldots s = c_2BD = .72 \times 4 \times 29.1 = 84 \text{ tons.}$$

$$\text{(11)} \quad \ldots\ldots\ldots a = \frac{2000}{S} \cdot s_t = \frac{2000}{20000} \times 84 = 8.4 \text{ square inches.}$$

$$\text{(10a)} \quad \ldots\ldots\ldots t = xd = .344 \times 29.1 = 10''$$

We have to make the stem of the beam wide enough to accommodate 8.4 square inches of steel, say $n = 12''$, and the girder is then designed as far as concerns the bending moment. Questions pertaining to shear, etc., will be considered later.

We can, if we desire, reduce the thickness t of the flange by increasing the depth d. While this operation is not always necessary, or even desirable, we will nevertheless continue the example to show the method of procedure.

If, then, we increase the depth from 29.1″ to, say, 35″, we get

$$\alpha = \frac{35}{29.1} = abt 1.2$$

the coefficient α indicating the proportionate increase in the depth. The value of β is next obtained by Table IX, remembering that

$$\frac{n}{b} = \tfrac{1}{4} = .25$$

the stem being 12″ wide and the flange 48″.

For $\frac{n}{b} = .25$ and $\alpha = 1.2$, Table IX gives $\beta = .43$; then, by the theory outlined for this case, Article 36, we have

The new depth $\quad D = \alpha d = 1.2 \times 29.1 = 35''$.

The new pull in steel $\quad s_b = \frac{s_a}{\alpha} = \frac{84}{1.2} = 70$ tons.

The new thickness of flange $\quad T = \beta t = .43 \times 10 = 4.3''$.

It is of course unnecessary to calculate the dimensions of the "minimum" beam first, as done here, unless we expressly desire to have these dimensions. Let us, for instance, again consider a given bending moment of 2,160 inch-tons; let us further select arbitrarily the width $B = 4' - 0''$, and let us finally choose the coefficient $\alpha = 1.2$, then, by Table IX, we get $\beta = .43$ for an estimated value of $\frac{n}{b} = .25$; and we also have $\alpha^2 = 1.44$. We may now find the dimensions directly, by

$$(13) \quad \ldots\ldots\ldots D = \alpha \cdot \frac{12.9}{c_1} \cdot \sqrt{\frac{M}{B}} = 1.2 \times \frac{12.9}{10.3} \times \sqrt{\frac{2160}{4}} = 35''$$

$$(14) \quad \ldots\ldots\ldots s_t = \frac{c_2}{\alpha^2} \cdot B \cdot D = \frac{.72}{1.44} \times 4 \times 35 = 70 \text{ tons.}$$

$$(15) \quad \ldots\ldots\ldots T = \frac{\beta}{\alpha} \cdot x \cdot D = \frac{.43}{1.2} \times .344 \times 35 = 4.3''$$

Figure 90 shows the resulting construction. The space Z

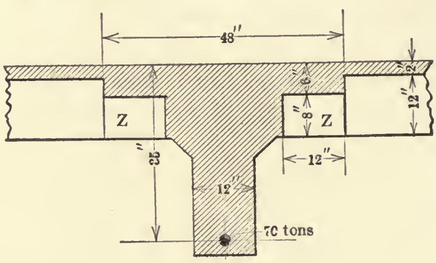


FIGURE 90.

under the flange may be taken up of tiles with less depth than those used for the balance of the floor, as long as the thickness of concrete over these lighter tiles is made equal to the thickness of flange just found, or thicker. In this way, the construction of the girder is lightened somewhat, and the form work may be made of the same light construction up to the face of the stem. A floor constructed in this manner also presents a tile surface to receive the plaster in all places except on the stem of the girder.

If we now wish to check our calculations, we may proceed as follows:

From Table IV, we get at once $c_3 = .89$, and the lever arm of stresses becomes

$$D_1 = c_3 D = .89 \times 35 = 31.2''$$

and we get the total pull in the steel

$$s_t = \frac{M}{D_1} = \frac{2160}{31.2} = 69.3 \text{ tons.}$$

The calculations above gave 70 tons. We now have to find the compressive resistance of the beam, and this we get as the difference between two items: A = the total resistance of the entire area above the neutral axis, and B = the section cut out between the neutral axis and the bottom of the flange. First we find the location of the neutral axis

$$xD = .344 \times 35 = 12.25''$$

and we get then

$$A = \tfrac{1}{2} \times 48'' \times 12.25'' \times 700 = 205{,}800 \text{ lbs.}$$

To find B, we must first find the concrete stress at the bottom of the flange, and by reference to Figure 86, we get

$$C_T = 700 \times \frac{12.25 - 4.3}{12.25} = 700 \times \frac{7.95}{12.25} = 454 \text{ lbs. per sq. inch.}$$

When determining B, we must remember that the "width of beam" is not 48″, but 48″ − 12″ = 36″, the 12″ being the width of the stem, and we get now

$$B = \tfrac{1}{2} \times 36 \times 7.95 \times 454 = 65{,}000 \text{ lbs.}$$

Then

$$A - B = 205{,}800 - 65{,}000 = 140{,}800 \text{ lbs.} = 70.4 \text{ tons.}$$

The total compression must of course equal the total resistance, and we see that our design is correct as this is the case. The slight difference between the 70 tons of the design and the 70.4 tons of the check is due to the inaccuracies of the slide rule and the various interpolations made, and is entirely too small to warrant further investigation.

48. Example 2. — Flat Slabs.

Given: Live plus dead load 500 lbs. per square foot. Span 12′ − 0″ between centers of support. Allowable stresses, concrete, 600 lbs., steel, 14,000 lbs. Continuous construction (q=10. −).

We get from Table II: c_1 = 10.1; Table III: c_2 = .71; and we can now proceed with the design using formulas (16) and (17), and we get

$$d = \frac{l}{c_1} \cdot \sqrt{\frac{w}{q}} = \frac{12.0}{10.1} \cdot \sqrt{\frac{500}{10}} = 8.4''$$

and $s_t = c_2 d = .71 \times 8.4 = 5.96$ tons per lin. ft. of width.

To the depth must now be added the amount of concrete required to properly protect the rods.

49. **To Find the Stresses in a Given Beam,** when the bending moment is known, requires a knowledge of the ratio S/C and of the ratio r. A simple mathematical analysis of the beam gives the first ratio, as we shall see presently; the value of r we cannot determine, so that it will have to be assumed in the same manner as when designing. The value 15 may very well be used.

We found above the expression (Article 35, Formula 12)

$$\beta = \alpha - \sqrt{\frac{\alpha^2 - 1}{1 - n/b}}.$$

while Article 36 gives the means for eliminating in this expression first β and α; by means of (6) and (11), c_2 and s_t are eliminated. The resulting equation may be solved for S/C, and gives:

$$\frac{S}{C} = r \cdot \frac{T \cdot H \cdot \left(1 - \frac{n}{b}\right) - VD + D \cdot \sqrt{(T+V)^2 - \frac{n}{b}(T^2 - 2\,VH)}}{2\,VD + \left(1 - \frac{n}{b}\right)T^2} \qquad (18)$$

The quantities entering on the right side of the equation mark are all known except r; in Figure 91 the several dimensions are

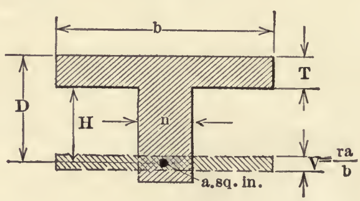


FIGURE 91.

shown; the value V is written as an abbreviation of ra/b, so that

$$V = \frac{ra}{b}$$

represents the thickness of an imaginary strip of concrete having the same width as the beam considered, and the same resistance as the tension steel; a and b are taken directly from the drawing, same as the other dimensions.

In special cases this formula is greatly simplified, although there is no difficulty whatever in using the formula given.

(1) For rectangular beams, or slabs, we have $n = b$ and $T = 0$ and we get

$$\frac{S}{C} = -\frac{r}{2} + \frac{1}{2}\sqrt{r^2 + \frac{2\,Drb}{a}} \tag{19}$$

(2) Disregarding the influence of the stem on the compression, as is sometimes done, we have $n = 0$ and get

$$\frac{S}{C} = \frac{2\,D - T}{\left(\frac{T}{r}\right) + \frac{2\,Da}{Tb}} \tag{20}$$

(3) If, in Formula 18, $T^2 = 2VH$, we find after some reduction

$$\frac{S}{C} = \frac{Tb}{2\,a} \tag{21}$$

The value of n/b disappears entirely, which evidently means that the neutral axis coincides with the bottom of the flange. The ratio $\frac{T^2}{2VH}$ is therefore the criterion of the section. If $= 1$, the neutral axis coincides with the bottom of the flange, if < 1, it falls below, if > 1, above the bottom of the flange.

If we now wish to determine the stresses in a given beam, we begin by selecting r, next we determine the value of the criterion, so that, if equal to unit, we use formula (21), while if larger than unit, we use formula (19), and if smaller, the original formula. Then the location of the neutral axis is calculated from

$$x = \frac{1}{1 + \frac{S}{C} \cdot \frac{1}{r}}$$

and the coefficient $c_3 = 1 - \frac{1}{3}\,x$ is determined from the value of x just found. The effective depth is then

$$d_1 = c_3 d$$

where d is the depth from ultimate compression fiber to the center of the steel. We have then

$$s_t = \frac{M}{d_1}$$

and, by (11)

$$S = \frac{2000}{a} \cdot s_t$$

We know now S and S/C, and it is a simple matter to determine C.

50. EXAMPLE 3. T-SECTION.

Given the beam shown in Figure 92, find the stresses, when the bending moment is 2,160 inch-tons. We have

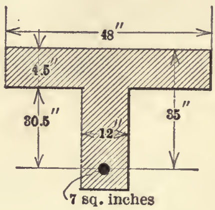


FIGURE 92.

$$V = \frac{ra}{b} = 15 \times \frac{7}{48} = 2.19$$

$$\frac{T^2}{2\,VH} = \frac{4.5 \times 4.5}{2 \times 2.19 \times 30.5} = \frac{20.25}{113.5}$$

which is evidently < 1. Using therefore formula (18) we find

$$\frac{S}{C} = 15 \times \frac{325.4}{168.4} = 28.9$$

Now $$x = \frac{1}{1 + \frac{S}{C} \cdot \frac{1}{r}} = \frac{1}{1 + \frac{28.9}{15}} = .342$$

and $$c_3 = 1 - \tfrac{1}{3}\,x = 0.886.$$

Hence $$d_1 = c_3 d = 0.886 \times 35 = 31''$$

Bending moment 2,160 inch-tons, then

$$s_t = \frac{2160}{31} = 69.6 \text{ tons.}$$

$$S = \frac{2000}{7} \cdot s_t = \frac{2000}{7} \times 69.6 = 20{,}100$$

$$C = \frac{20100}{28.9} = 695.$$

or about 20,000 and 700 lbs./square inch for steel and concrete, respectively.

51. EXAMPLE 4. T-SECTION. Special case.

Given the beam shown in Figure 93.

We have $$V = \frac{ra}{b} = 15 \times \frac{8.4}{48} = 2.62.$$

$$\frac{T^2}{2\,VH} = \frac{10 \times 10}{2 \times 2.62 \times 19.1} = \frac{100}{100} = 1.$$

Use formula (21) which gives

$$\frac{S}{C} = \frac{10 \times 48}{2 \times 8.4} = 28.6$$

The balance of the calculations may now be continued exactly as in the preceding example.

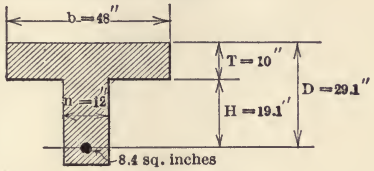


FIGURE 93.

52. EXAMPLE 5. RECTANGULAR BEAM. Slabs.

Given the rectangular beam shown in Figure 94; we use formula (19) which gives

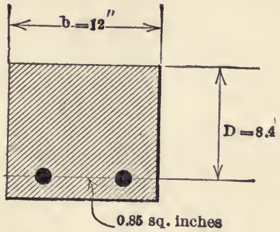


FIGURE 94.

$$\frac{S}{C} = -\frac{15}{2} + \frac{1}{2} \cdot \sqrt{225 + \frac{2 \times 8.4 \times 15 \times 12}{0.85}} = 23.5.$$

CHAPTER VII

TRANSVERSE STRESSES.—U-BARS

53. In addition to the longitudinal stresses examined in the preceding articles, transverse stresses exist in reinforced concrete beams as well as in beams of other materials. But the transverse stresses are different in trusses and in solid beams: in the *truss*, each individual member is stressed in its longitudinal direction only, and there is no shear. In the *solid beam*, longitudinal stresses exist in the top and bottom chords or fibers, and the web is then subject to shear stresses both longitudinally and transversely. In special cases these shear stresses may vanish, as for instance in the I-shaped steel beam of variable depth, when the ratio

$$\frac{\text{bending moment at any point}}{\text{depth at the same point}}$$

is a constant. This is the case in a parabolic girder loaded over its entire length with a uniformly distributed load.

54. In view of this difference between trussed beams and solid beams, it becomes necessary to decide whether to treat the reinforced concrete beam as the one or the other. To the eye a reinforced concrete beam certainly appears solid enough, and such is indeed the case when the beam is first made and the load is being put on. But when the load reaches a certain intensity the "solidity" of the beam is destroyed. Slight cracks soon become evident, at least when arrangement has been made to observe them, and that under loads corresponding to a steel stress of from 4,000 to 6,000 lbs./square inch, or a concrete stress of 350 lbs./square inch. It follows that under the ordinary working load our reinforced concrete beam is perforated with cracks extending from the bottom fiber up toward the neutral axis, without quite reaching the neutral axis, so that, under any circumstances, the beam is certainly not a "solid" beam. These hair cracks have been noted by all who have

taken the trouble to look for them with but one exception (Considère); they are not an occasional occurrence, but a universally recognized phenomenon of the greatest importance for our understanding of the stresses within a reinforced concrete beam. The presence of these cracks is accounted for by the simple fact that concrete is unable to stretch as much as steel before cracking, so that, under a certain load, the concrete refuses to follow the steel in its elongation and goes to pieces. The cracks of this class appear throughout the length of the beam, fairly uniformly spaced, and increase in size with increasing load.

55. The crack of course is an open space existing between surfaces which at some earlier time were in close contact and united. We must now understand as a fundamental principle that stresses cannot be transmitted through open cracks. Compression may be transmitted through a contact only, and friction may exist on surfaces pressed together, but no kind of stress will jump across an open space. It follows that shear in the ordinary sense of the word cannot exist in a reinforced concrete beam loaded above a certain limit, because the nature of shear requires equal intensity on a horizontal and a vertical plane, and this is of course impossible when the beam has vertical cracks. Or, we may simply say that the vertical shear cannot exist in the crack itself. Where a crack occurs there is therefore nothing but the compression flange and the tension steel to carry the shear, — a distribution of the shear which is, to say the least, not easily reconciled with current ideas of shear in solid beams.

56. Entirely different from these hair cracks are the much larger, pronounced failure cracks which predict the approach-

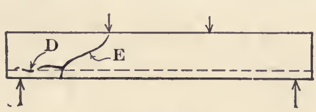


FIGURE 95.

ing collapse of the test beam. If located at or near the point of maximum bending moment, they are undoubtedly due to excessive elongation of the steel disclosing a failure by tension in the steel; if near the end, the crack usually takes the shape shown in Figure 95, either with or without the horizontal crack

D. The vertical crack *E* is wide open, especially at the bottom, decreasing in width as it approaches the top of the beam. As the steel stress at this point certainly cannot exceed the steel stress at the point of maximum bending moment, this crack is not due to excessive tensile stresses in the steel. It must be due to sliding of the reinforcement: the steel is pulling out of the end of the beam at the same time bursting its concrete envelope, and causing the horizontal crack. Let it be understood that no amount of shear will cause a *gaping* crack, but once sliding sets in and causes the vertical crack, it is clear that the one end of the beam will be compelled to revolve around the other end, causing in the first place the double-curved line of cleavage, and, secondly, great friction on the surfaces of contact.

57. The above remarks lead to the conclusion that a concrete beam is a solid beam up to a certain load at which point the tensile resistance of the concrete is exhausted, and a readjust-

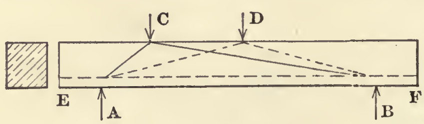


FIGURE 96.

ment of stresses takes place within the beam. This readjustment is different for different types of beams. In a rectangular beam (Figure 96) we may well assume that the compression follows lines as *AC* and *BC* when the load is placed

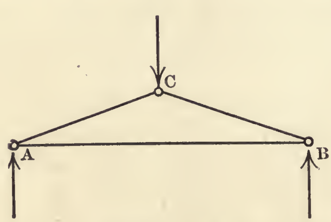


FIGURE 97.

at *C;* if the load moves to *D*, the lines change to *AD* and *DB*. Under these circumstances there is no shear, at least not in the ordinary sense of the word. We may compare a system of this kind to a triangular frame with hinged corners (Figure 97). The chords AC and BC will be in compression, and the chord AB in tension, hence at A and B the hinges are subject to severe stresses. The same is the case at A and B in the reinforced concrete beam, so that the " length of embedment " AE and BF in Figure 96 must be made long enough to prevent sliding of the rod. The shear existing in a system of this kind is that negligible quantity caused by the stiffness of the system as a whole, a kind of friction caused by the lack of flexibility at the supposed hinges.

The system ABC is an equilibrium curve for the load C and the reactions A and B; this same argument would of course hold true for any number of forces, or even for uniformly distributed loads, in which latter case the compression curve would be a continuously arched curve from A to B (when the load covers the entire span). But if the beam under consideration is a T-beam instead of a rectangular beam it becomes impossible to make the compression line curve down to the supporting points, except for a width equal to that of the stem. A T-beam (Figure 98*a*) may be considered as consisting of two

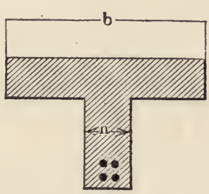


FIGURE 98*a*.

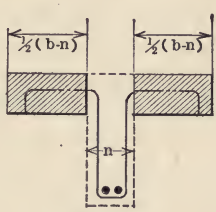


FIGURE 98*b*.

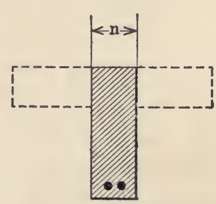


FIGURE 98*c*.

beams side by side; a T-beam proper (Figure 98*b*) and a rectangular beam (Figure 98*c*). In this rectangular portion it is quite possible for the compression lines to dip down at the supports, but not so for the T-beam portion, there being no concrete left to carry the stresses down to the steel. This leads to the idea of bending the steel up over the support to meet the compression flange, reversing the conditions shown in Figure 96.

58. Let us consider a portion of a reinforced concrete beam between two points a and b (Figure 99). The bending moment at a is denoted by M_a, the distance between center of com-

pression and center of tension by d_a, then the total pull in the steel at a is

$$s_a = \frac{M_a}{d_a}$$

and at b

$$s_b = \frac{M_b}{d_b}$$

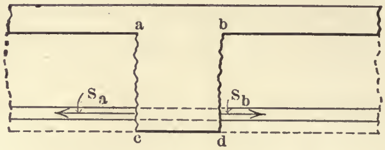


FIGURE 99.

The difference between these two is

$$\Delta s = s_a - s_b = \frac{M_a}{d_a} - \frac{M_b}{d_b}.$$

The prefix Δ simply denotes the difference in the item considered so that Δs means the variation of s between the points in question.

It is now evident that the portion *abcd* is subject to two pulling forces acting near its lower end *cd:* one force s_a pulling toward the left, another pulling toward the right, s_b. If s_a is larger than s_b, the end *cd* must have a tendency to move toward the left in precisely the same manner as if pulled that way by a force equal to the difference of the two pulling forces; we may therefore consider the force $\Delta s = s_a - s_b$ as acting alone. This condition is represented in Figure 100, and this diagram

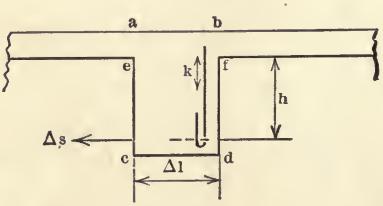


FIGURE 100.

shows at once that *cdef* is a cantilever fixed at its base *ef*, and loaded near its end with a load Δs. The depth *ce* we do not know at the present time; let us indicate this unknown quantity by h. The bending moment on the cantilever is then $h\,\Delta s$; the arm of "the couple of stresses" in the cantilever is $c_3\,\Delta l$; hence if a vertical reinforcing rod is disposed near bd the pull on this rod becomes

$$k = \frac{h\Delta s}{c_3\Delta l} \tag{22}$$

But this pull k can exist only when counterbalanced by a corresponding compression, so that the beam becomes a trussed beam as shown in Figure 101. The vertical reinforcement

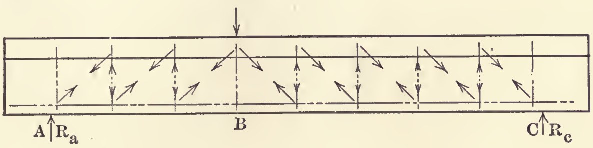


FIGURE 101.

designed in this manner is usually made of a bar bent to U-shape and circling the main tension rod (Figure 102*a*, *b*); they are therefore called U-bars or stirrups. The U-bar is unneces-

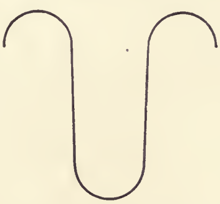


FIGURE 102*a*.

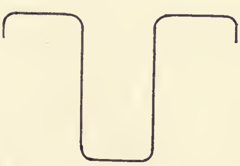


FIGURE 102*b*.

sary when $k = 0$, which is always the case when $\Delta s = 0$: i.e., when either the tension chord, or the compression chord, or both together, follow the equilibrium curve. As shown above, this is always the case in a rectangular beam with well-anchored reinforcement, and it is also the case for such parts of a T-beam in which the reinforcement is bent up to follow the equilibrium curve. In all other cases k has a definite value. For straight reinforcement and straight top chord, we have (Figure 103):

$$d_a = d_b = c_3 d$$

hence $$\Delta s = \frac{M_a - M_b}{c_3 d}$$

where $$M_a = Ra - \Sigma Pp$$
and $$M_b = R\,(a - \Delta l) - \Sigma P\,(p - \Delta l)$$
hence $$M_a - M_b = \Delta l\,(R - \Sigma P)$$
or $$\Delta s = \frac{\Delta l}{c_3 d}\,(R - \Sigma P)$$
and $$k = \frac{h}{c_3^2 d}\,(R - \Sigma P).$$

It is now easy to understand that the length h cannot exceed

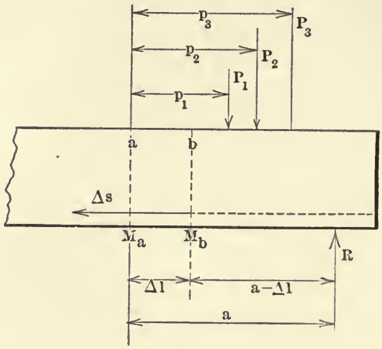


FIGURE 103.

the distance from the center of the steel to the neutral axis. This gives

$$\frac{h}{c_3^2 d} = \frac{(1 - x)d}{(1 - \frac{1}{3}\,x)^2 d} = \frac{1 - x}{(1 - \frac{1}{3}\,x)^2}$$

The value of this expression cannot exceed unit; for ordinary cases its value is about three-fourths. Hence the maximum possible value of k, for the conditions named, is:

$$k_{max} = R - \Sigma P. \tag{23}$$

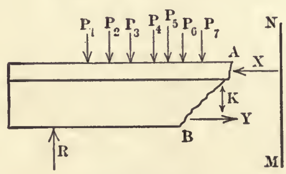


FIGURE 104.

59. It is now interesting to note that this same expression may be obtained directly in the simplest manner. Let, in Figure 104, the section AB remove the right end of the beam

leaving the main tension bar and the U-bar projecting. The stress-resultants acting upon AB are then, when the chords are parallel: the horizontal compression X, the horizontal pull Y, and the vertical force k in the U-bar. The loads are P_1, P_2, etc., and the reaction R. If we now project on a vertical line MN, the horizontal stresses vanish and we have

$$R - (P_1 + P_2 + P_3 + \ldots\ldots) = k$$

or

$$k = R - \Sigma P.$$

60. The beam with straight top- and bottom-chords is an exception. Usually the stem of a T-beam may be considered as in equilibrium, and in addition some of the bars are bent up in the T-beam portion to approximate the equilibrium curve, so that a material reduction in the value of k takes place in all practical beams. With the notations of Figure 98 we have, on account of the stem, a reduction equal to

$$\frac{b-n}{b} \quad \text{hence} \quad k = \frac{b-n}{b}[R - \Sigma P]$$

If out of the total number x of bars in the T-beam, a certain number y follow the equilibrium curve, we have a further reduction equal to

$$\frac{x-y}{x} \quad \text{hence} \quad k = \frac{x-y}{x} \cdot \frac{b-n}{b} \cdot [R - \Sigma P]. \tag{24}$$

Thus, if $b = 48''$ and $n = 12''$, we have

$$\frac{b-n}{b} = \frac{48-12}{48} = \frac{3}{4}$$

so that, out of a total number of say eight bars, the six belong to the T-beam. If out of these six, two are bent as required, we have $x = 6$ and $y = 2$, hence

$$k = \tfrac{4}{6} \times \tfrac{3}{4} \times [R - \Sigma P] = \tfrac{1}{2} \cdot [R - \Sigma P]$$

61. Thus, in order to calculate the stress on the U-bars, it becomes necessary to know the properties of the curve of equilibrium for the system. When the loads are stationary, the curve is drawn as a force polygon to the actual loads and reactions. For a uniform load, covering the entire span, this curve is a parabola; it is not practical to bend the bars to this shape, but it may be closely approximated by a system of bars with straight portions between the several bents. A uniformly

distributed, moving load has no definite curve of equilibrium, so that in that case the most dangerous position of the load must be found and the U-bars proportioned according to Formula 23 above, while the bent bars are arranged to meet the requirements of some particular type of loading, for instance, the total load. Similarly, concentrated loads may be either stationary or moving. In buildings the concentrated loads are usually stationary. The given load is a uniform load, so that the beams are loaded as explained above; these beams in turn frame into the girders, one, two, or three beams to each span, and these concentrated beam loads are stationary. It is a simple matter to bend the main tension bars to conform to this type of loading; examples are given in Figures 105 and 106.

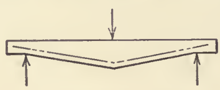


FIGURE 105.

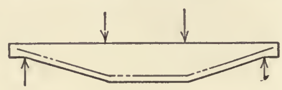


FIGURE 106.

The moving concentrated load is usually found only in structures like highway bridges, subject to steam-roller traffic, in crane-track girders, etc. In such cases the live load is large in proportion to the dead weight of structure and covering, so that the T-beams are usually not economical structures for this class of girders. They may be constructed by using the adequate number of U-bars; or rectangular beams may be used of the required cross-section.

62. The problem of designing a T-beam under a uniform load confronts the reinforced concrete designer every day. It is customary to consider the load as covering the entire span, except in cases where it is expressly stipulated that the most dangerous position of the load shall form the basis for the calculation of the U-bars. Arguments may be advanced *pro et con.*, — usually the load specified is a maximum load which seldom, if ever, covers the entire beam, and the designer will have to use his best judgment as to what constitutes proper practice in each individual case. It is hardly necessary to say that in other lines of engineering the most dangerous condition is always considered in making the calculations as a matter of course, and there is no reason why other professional ethics should prevail when dealing with reinforced concrete.

In Figure 107 the moment-curve is shown corresponding to a uniformly distributed load covering the entire span. The maximum moment is taken as unit, and the several ordinates of the curve are given under the assumption that there is no continuity. The reinforcement must be made to conform to

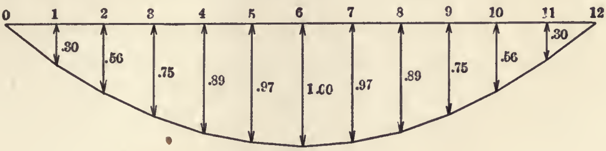


FIGURE 107.

this curve as closely as possible, hence we see that at points 3 and 9, only $\frac{3}{4}$ of the total number of bars is required, at 2 and 10, slightly more than $\frac{1}{2}$, and less than $\frac{1}{3}$ is required at points 1 and 11. The quota of bars not required may and should be bent up at the points specified, provided that no other kinds of loading can occur. In Figure 108 the corresponding curve is

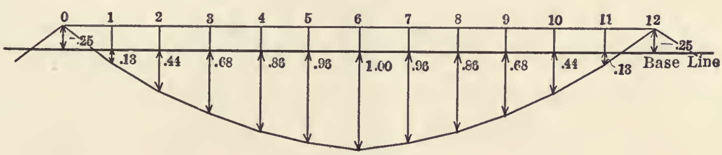


FIGURE 108.

shown when the beam is considered as continuous, with $q = 10$.

63. The entire theory outlined for the calculation of the U-bars is based upon the assumption that sliding of the steel cannot take place. In such cases where the anchorage beyond or at the supports is insufficient to prevent sliding of the main tension bars the factor of reduction must be decreased, so that a correspondingly larger amount of vertical reinforcement is used for the U-bars. In the present state of our knowledge this must be taken care of by judgment alone, there being no way of calculating a beam with inefficient anchorage. It must here be sufficient to point to the fact that the U-bars retard the sliding of the reinforcement, and that, for that reason, light U-bars should always be used even in cases where the theoretical considerations show that they may be dispensed with. This applies particularly to rectangular beams.

64. **Spacing of the U-Bars.** It will be noted that the entire line of argument advanced in the preceding paragraph is based principally upon the inability of the concrete to resist tensile stresses, and that the entire problem finally resolves itself into one of tension carried entirely on the steel, and compression carried entirely on the concrete. The word " shear " is referred to incidentally only, and this is a natural consequence of the fundamental principle of disregarding the tensile stresses in the concrete. As this development leads to rather important results, it may be well to consider these matters a little more in detail.

65. Figure 109 shows the simplest conceivable system of material units, i.e., three particles, A, B, and C. Whatever the nature of the force uniting these particles, if the particle C is moved to the position D through the influence of some external force, the displacement CD represents in all cases the result of the influence of that force and is called the "shear deformation" if parallel with the line AB. It is readily seen, however, that the more direct and more readily understood deformations are (1) the lengthening of AC to AD, and (2) the shortening of BC to BD. Hence this shear deformation CD is nothing but the resultant of the deformations along the original lines AC and BC, and we perceive that even in the most complicated system of particles any deformation may be reduced to a system of lengthenings and shortenings, that is, tension and compression, if we speak of stresses instead of deformations. The word " shear," therefore, has no real or material meaning, except as a pure figure of speech to express in one short word a rather intricate condition of tensile and compressive relations, in precisely the same manner as the word " bending moment " is used to indicate a mathematical conception of the mutual condition of a number of forces acting upon a beam. Needless to say that nobody has ever seen, or will ever see, a bending moment in the realm of things as they are, and that whoever undertakes to explain the so-called " shear stresses " in a solid body will ultimately have to account for pure tensional and compressional stresses.

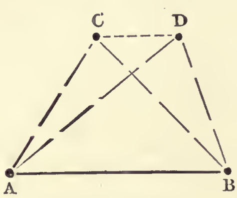


FIGURE 109.

66. If two material bodies are in contact, the stresses acting *in the contact surface* are termed frictional stresses which, as far as the materials themselves are concerned, are compressive stresses with no possibility of accompanying tensile stresses in the direction perpendicular to the contact surface.

67. Of the nature and extent of frictional stresses we know next to nothing. A force acting parallel with the contact surface will cause sliding of one body in relation to the other; if the force is inclined, the sliding becomes increasingly difficult as the angle of the force increases, and the sliding becomes impossible when the angle at which the force acts exceeds the "angle of friction," which has a definite value for each material, depending in part upon the character of the surface. For concrete upon concrete, this angle appears to be near 41°.

68. In certain types of reinforced concrete construction the floor beams are not made in one continuous operation with the floor slab resting upon the beams, and U-bars or similar mechanical devices are then resorted to in order to tie the slab and stem together, and to so unite them that they may be considered as acting as one piece. In this case, the slab would form the upper flange of a T-beam, and in order to insure this action, *sliding* between flange and stem must be prevented. Figure 110 represents a portion of a beam, the lines AC, CD, and CB

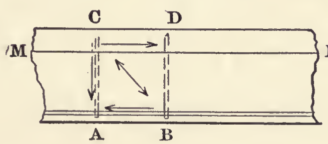


FIGURE 110.

indicate the directions of the principal stresses. If now the line of diagonal compression BC is inclined so that the angle BCD is less than the angle of friction, the flange would slide in relation to the stem, on account of the joint along the line MM; — the U-bars AC and BD would resist this tendency by virtue of their "shear" resistance (and this resistance we know is very small, and cannot exceed the compressive edge resistance of the concrete; see Figures 111, 112, where the black areas indicate the crushed concrete). If, on the other hand, the angle BCD is larger than the angle of friction, then there can be no sliding, and therefore no shear stresses on the U-bars, which will act directly in tension as described above. The rule derived from this argument may be briefly expressed thus: The spacing of the U-bars must not exceed the depth of the beam, in which case the angle of forces would be about 45°.

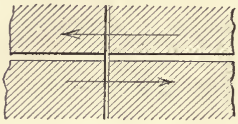


FIGURE 111.

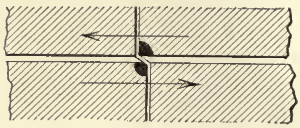


FIGURE 112.

69. If we now turn to the T-beam manufactured in one continuous operation, where no separation exists between stem and slab, we note that, theoretically at least, this beam is in the same condition as the one just considered, owing to the original assumption whereby the tensile stresses in the concrete are considered as non-existing. Each and every horizontal stratum must be considered as isolated and influenced by its neighbor through the medium of frictional resistance only, and the direction of the diagonal compression must be such that no sliding can take place. The same rule must therefore be imposed in this case.

But this rule gives the maximum spacing possible: owing to the usual considerations of a margin of safety, the spacing must be made smaller, and we would therefore recommend that the spacing of the U-bars must in no case exceed one-half the effective depth of the beam.

70. **Tensile Stresses in Concrete Disregarded.** The ready-made reinforced concrete beam formulas now in common use are derived under the apparent assumption that the steel reinforcement takes all the tensile stresses, and this is also the case in this book. In reality, we cannot wholly disregard these tensile stresses in the concrete, or, at least, we cannot deny their existence, because if we did, we would also rob the concrete of its cohesion, and we would have a granular mass such as sand or crushed stone, wholly unsuitable for our purpose. The true statement is that we disregard the tensile stresses in certain directions and for certain purposes. In this book, we have considered the concrete as fractured vertically along the planes

of the U-bars (1) because the cracks in probability will appear in the weakest plane, there being less concrete to resist the tension where the concrete is displaced by the steel of the U-bar; (2) because the U-bar encircling the main tension rod in a measure acts as a washer on the rod, causing the somewhat resilient concrete to crack immediately behind this point of gripping; and (3) because such tests as throw any light upon the location of the cracks indicate that they occur very largely at just these points.

71. NOTE: For the gripping action of a loose U-bar encircling the tension rod, see Mörsch, page 47.

For the location of the cracks, see the same book, page 155.[1]

72. We have also considered the stem of our T-beam as composed of horizontal layers acting upon one another by contact only, and thereby determined the spacing of the U-bars. But between these vertical and horizontal lines of weakness, we have assumed the concrete to be solid. Hence, we have assigned to the concrete a certain amount of tensile resistance in certain locations and directions.

73. It follows that with increasing loads the compressive stresses in the beam do not increase as rapidly as the load, especially not in beams where the slab and the stem are separately manufactured. In such beams, the compression at rupture must in many cases be *uniformly distributed* over the entire compressive zone, and we find here the explanation of the fact sometimes observed that the compressive strength of concrete is much higher in a beam test than in a cube test. An analysis of these conditions would be interesting and of great value practically.

74. **Details of Reinforcement.** The various arguments advanced above will lead to rational design of the steel if consistently applied, and there is but little new to add. The great principle in all beam construction is that there is a compres-

[1] Mörsch: Concrete Steel Construction, 1909. While the cracks do not all occur at the U-bars, the *tendency* is fairly pronounced, especially in the beams with U-bars in one half only, see Figure 149, Beam V; Figure 153, Beam VIII; Fig. 154, Beam IX; Fig. 157, Beam X; and compare the cracks in the U-bar end with those of the other end of the same beam. The drawings of all these beams show them just before final collapse, while our calculations have reference to a much earlier stage, viz., under the working load, or at the most a load not more than twice the working load.

sion and a tension, separate from one another, but with horizontal projections of equal intensity or magnitude, provided the loads are vertical. Whatever the arrangement, the compression and tension must ultimately meet one another and annihilate one another, whether this takes place gradually by increments, as in the plate girder of constant depth; or in one operation, as in the King truss, where the tension chord meets the compression chord at the ends of the beam; or in a number of places, all well defined, as in the Howe truss. We have seen that the rectangular beam is somewhat similar to the King truss, and that the T-beam is very similar to the Howe truss; we have also pointed out that the theory of stress-transmission by gradual increments is not tenable under high loads owing to the slight tensile resistance of the concrete. We must assume that the sooner the compression and the tension are brought to annihilate one another the better will our beams withstand the loads, hence the necessity of bending the rods up as soon as possible, and the desirability of closely spaced U-bars. A simple and effective way of bending the bars is shown in Figure 113. The point of bending should be deter-

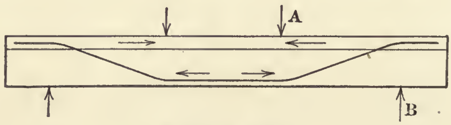


FIGURE 113.

mined by the bending moment, so that there is steel enough to meet the requirements at all points. In this beam and the following we must suppose that there are some straight bars, but these are not shown in the figures. Hence the principal stresses in Figure 113, disregarding the straight bars, are: a constant compression along the slab, a constant tension in the rod, and certain vertical resultants. The rod has a curve under the load A, against which the concrete is pressing. The resultant of all these pressures should go through the point of application of A, hence the rod should be bent to a circle with center in the point of application. The same applies to the reaction, B, and in addition the rod should be extended beyond the support to develop the full adhesive resistance.

A somewhat more complicated method is shown in Figure114 where there are two systems of bent rods (aside from some straight ones). The " first " rod, AC, is curved under the load P for the reasons explained above; in addition, the resultants C and D must be made to meet one another in the same point and with the same direction and same force. Hence the number of rods in each chord should be the same. The length of rod in compression flange ab should be sufficient to develop the full strength of the bond, in the same way as for the " second "

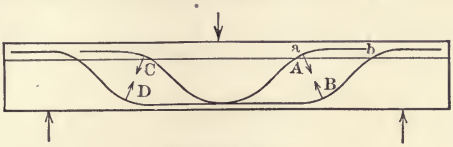


FIGURE 114.

chord over the point of support. The slope or angle of the bent bars would seem to be of no importance; but many authorities are of another opinion and recommend an angle of about 45 degrees. (In practice the bars are seldom bent to such large radii as shown in Figure 114, this diagram being purposely exaggerated.)

75. The shape of the U-bars should be as shown in Figure 102*a*, *b*, with curved top and bottom, and hooked over. The downward projection of the end makes it easy to support the U-bar on the form work, and the entire U-bar is firmly anchored against sliding, both top and bottom: the top on account of the curves, the bottom because it passes around the reinforcement. The direction of the U-bar should be vertical. The sloping or slanting U-bar is said to strip the concrete away from the tension rod, as we might expect if our theory is correct, and it does not give as efficient reinforcement in the small cantilevers as the vertical U-bar. Round U-bars appear to be better than flat bars; but there is a great amount of information along this and similar lines which will have to be furnished before reinforced concrete design can be perfected. But our lack of information in this and similar cases is not different from that existing in other lines of engineering.

76. When we now finally combine all these elements to one beam, Figure 115, we have a structure of a very complicated nature, and we must ask ourselves if all these stresses can travel through and between one another as here assumed without

upsetting our calculations and assumptions entirely. To this we must answer that we do not know, but if we compare our problem with those met in other lines of engineering we must admit that there is no fundamental difference between the difficulties. Thus a combination of two simple Pratt trusses is treated as if the two trusses were really present individually instead of combined into one structure, and many other instances

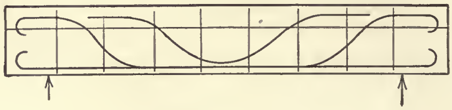


FIGURE 115.

could be cited to show that we often have to dissolve a structure into its apparent elements in order to solve its problems. Assuming the reinforced concrete beam to be similar to a Howe truss, as here proposed, seems to be no more of a mistake than to assume the connections in a riveted truss to be frictionless, movable joints. But the approximations made in steel construction are so old that they seem almost part and parcel of the art, while the comparatively new assumptions made for reinforced concrete have hardly had time to solidify, and they are therefore supposed to be of a more questionable nature than the older ones, which have indeed had the profit of the test of time. Yet there is a number of reinforced concrete buildings about thirty years old which stand up as well as anybody could wish, and the modern steel sky-scraper is of no older date.

CHAPTER VIII

APPLICATIONS OF THE BENDING THEORY

77. **Continuity of Reinforced Concrete Beams.** The difference between the beam with simple supports and the continuous beam is that the continuous beam is subject to a "reverse" bending moment over the support, while in the simple beam there is no such reverse moment. The cantilever beam is an example of the beam in which only reverse moments exist, and as we have found it feasible to construct reinforced concrete cantilevers we cannot deny that continuity *may* exist in reinforced concrete beams. In fact, unless special precautions are taken to eliminate reverse moments over the supports, we know that continuity *must* exist and should be taken into account. The question is then: to what extent are the ends of a reinforced concrete beam restrained? When this question is answered we must make the beam strong enough to resist the bending moment at the column, and then it is a matter for further investigation to decide in how far the beam is actually benefited by the restraint to such extent, that the moment at the middle of the beam may be reduced.

In Figure 116 a beam is shown in which the ends are per-

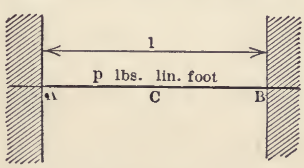


FIGURE 116.

fectly restrained, and where the uniform load covers the entire span. The bending moments are over the supports:

$$M_A = M_B = \tfrac{1}{12}\, pl^2$$

at the center: $$M_C = \tfrac{1}{24}\, pl^2.$$

Hence $$M_A + M_C = M_B + M_C = \left(\tfrac{1}{12} + \tfrac{1}{24}\right) pl^2 = \tfrac{1}{8}\, pl^2;$$

or: the total amount of bending moment to be taken care of in the beam with " built in " ends is the same as in a simply supported beam. The bending moment carried by a reinforced concrete beam is

$$m = \text{a constant} \times bd^2 \text{ (Formula 8);}$$

hence for constant depth the allowable bending moment is directly proportional to the width b. At C, Figure 116, the width is $= b$, but at the support where the reverse moment must be taken care of, the width of beam is only that of the stem $= n$. Hence if we assign a moment M_c to the middle of the beam, the end will only carry a moment

$$M_A = M_B = \frac{n}{b} M_C$$

so that
$$M_A + M_C = \frac{n}{b} M_C + M_C = \frac{1}{8} pl^2;$$

or
$$M_C = \frac{b}{n+b} \cdot \frac{1}{8} \cdot pl^2.$$

For
$$\frac{n}{b} = \frac{1}{4}$$

we have
$$M_C = \tfrac{4}{5} \cdot \tfrac{1}{8} \cdot pl^2 = \tfrac{1}{10} pl^2$$

and
$$M_A = M_B = \tfrac{1}{4} \cdot \tfrac{1}{10} pl^2 = \tfrac{1}{40} pl^2.$$

For
$$\frac{n}{b} = \frac{1}{6}$$

we have
$$M_C = \tfrac{6}{7} \cdot \tfrac{1}{8} \cdot pl^2 = \tfrac{1}{9.3} pl^2$$

and
$$M_A = M_B = \tfrac{1}{6} \cdot \tfrac{1}{9.3} pl^2 = \tfrac{1}{56} pl^2.$$

The moment at the center of the span, in the case of a T-beam, will therefore be about

$$\frac{1}{10} pl^2$$

and at the end
$$\frac{1}{40} pl^2.$$

If greater depth is provided near the support the reverse moment may be increased and the moment at the center of the span may be decreased a corresponding amount.[1] In Europe it is quite

[1] Attention is called to the obvious fact that no degree of "restraint" can be allowed at wall ends; this is especially true for beams resting in brick work.

common to make the beams and girders deeper at the columns; in America the beams and girders are usually of the same depth throughout. The American practice is to be preferred, because the continuous effect depends entirely upon the stiffness of the supports: the slightest yielding of the footings, or even the compressibility of the columns may destroy the continuity entirely, and too much dependence upon the continuous effect may lead to serious trouble.

In a slab, the depth and the " width of beam " is the same at the middle of the span and at the supports. If the supports are unyielding there may be some excuse for allowing a higher degree of continuity for slabs than for beams; the more so because tests on reinforced concrete buildings point distinctly to such effects. Let us assume a degree of continuity leading to the following bending moment:

$$M_c = \frac{1}{q} p\, l^2.$$

In Figure 117 the equivalent system of construction is shown

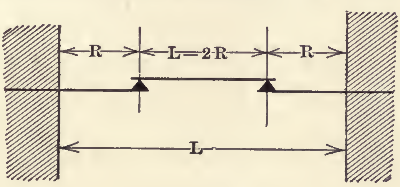


FIGURE 117.

in which the center portion is considered as a simple beam resting upon cantilevers of span R. We have then

$$(L - 2R)^2 = \frac{8}{q} L^2, \quad \text{hence} \quad R = \tfrac{1}{2} L \left(1 - \sqrt{\frac{8}{q}}\right) \qquad (25)$$

It is hardly necessary to say that we have no absolute certainty that the slab will adjust itself to conform to this arbitrary division of the bending moment. Yet if the cantilever is made strong enough to carry *its* load, and the central portion strong enough to carry *its* share, it is difficult to see why such a system should not be perfectly safe. Other assumptions may be made and carried through in the same manner; this analysis will be used later for the calculation of the " mushroom " system as invented by Mr. Turner.

The formulas usually given for continuous beams depend upon the factor EI. The value of I for a reinforced concrete beam is not a constant; in Article 79 we shall consider this in detail. We will find that the moment of inertia depends upon the maximum unit stresses in the point considered, and we cannot expect these stresses to be uniform throughout the length of the beam. The usual application of the formulas for continuous beams presupposes that the moment of inertia is constant throughout the length of beam, and we cannot therefore apply the formulas used for homogeneous beams to the reinforced concrete beams with any degree of certainty.

78. While, then, the exact degree of continuity cannot be determined, continuity does nevertheless exist in many cases if not in all, and the stresses thus created must be taken care of. These are, primarily, tensile stresses over the supports, requiring reinforcement in the top of the girders over the columns, in the beams over the girders, in the slabs over the beams. The top bars may be loose bars, but it is rather difficult to maintain such bars in their proper position; the bent-up bars may be utilized as top reinforcement with good results, especially as they extend a distance into the next bay in any case. It is evident from the remarks made above that the top reinforcement over the support should not be less than 25 per cent. of the bottom reinforcement; usually more bars are bent up, but they need not all extend as far beyond the support as the bars designed to resist the reverse moment. For a uniform load covering the entire span, the point of inflexion is evidently determined by the Formula 25:

$$R = \tfrac{1}{2} L \left(1 - \sqrt{\frac{8}{q}}\right)$$

so that 25 per cent. of steel mentioned above should be carried at least that distance out from the center of the support. The bars must be embedded in a sufficient amount of concrete to develop the bond, not less than four diameters from the face of the concrete, or, if closer to the face of the concrete, they should be provided with inverted U-bars. The stress on these U-bars cannot be calculated, — it is their presence rather than their strength which benefits the beam.

79. **Moment of Inertia.** The moment of inertia in a rein-

forced concrete beam is of interest only because certain problems connected with continuity of the beam, deflection, etc., cannot be solved except through a knowledge of its value. The expression given below is of indirect value only, showing that the ordinary formulas for continuity do not apply to reinforced concrete beams, because the moment of inertia is not a constant for the length of the beam, as is usually assumed in the solution of such problems.

The moment of inertia with reference to the neutral axis may be found as the sum of two moments: I_1, referring to the concrete above the neutral axis, and I_2, referring to the steel below the neutral axis, the concrete below this line being disregarded as usual. We have then (Figure 118)

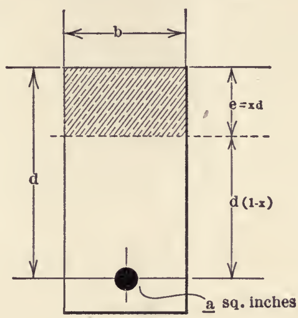


FIGURE 118.

$$I_1 = \tfrac{1}{3}\, b\, (xd)^3$$

and

$$I_2 = rad^2\, (1 - x)^2$$

the steel being considered as equal to ra square inches of concrete. But according to the formulas given in Articles 25 ff. we have

$$\tfrac{1}{2}\, Cxdb = aS$$

or

$$a = \frac{xdb}{2} \cdot \frac{C}{S} = \frac{x^2db}{2\,(1 - x)\, r};$$

hence (after some reduction):

$$I = I_1 + I_2 = \tfrac{1}{2}\, bd^3\, (1 - \tfrac{1}{3}\, x)\, x^2 \qquad (26)$$

By means of Formula 5 in Article 27 the expressions derived above for d and s_t, etc., may now be verified. The real importance of Expression 26 is, however, that it shows that the moment of inertia depends upon the location of the neutral axis which again changes with the stresses in the various points of the beam.

80. **Beams with Reinforcement in the Compression Side.** Sometimes it is found impossible to make the compression flange of the beam wide enough to bring the concrete stress down to the allowable maximum. In that case some engineers use compression reinforcement, but as a matter of fact, our knowledge of the properties of such beams is very slight, and there is grave doubt as to the advisability of using this method of construction in important cases. The calculations are simple: to the bending moment sustained by the beam with its ordinary amount of reinforcement is added another bending moment due to extra reinforcement in top and bottom, this latter calculated as for an ordinary steel beam, but with quite low stresses (not to exceed 10,000 lbs./square inch). The compression bars must be laced carefully to the tension bars, but under any circumstances it seems hardly possible to provide properly for the excessive shear stresses set up in this kind of beams. A steel I-beam is cheaper and better in places where this kind of construction is actually necessary.

81. **Combined Bending and Compression.** The section is best designed by trial. In the case of an *arch ring*, the section is rectangular, and the symmetrical reinforcement is of small area compared with the concrete section. The bars on the compression side must therefore be disregarded, as it would require too many hoops to make this reinforcement effective in compression. We must select the depth and the reinforcement by judgment; the stresses due to the bending moment alone are then easily found by Formula 19 in Article 49. Let Figure 119 represent the section; let C_m and S_m denote the stresses just found due to the moment alone.

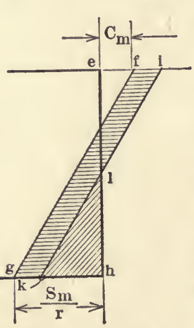


FIGURE 119.

If now in Figure 119 gh is made equal to S_m/r and ef is made equal to C_m, then the line gf will represent the distribution of stresses on the section due to the bending moment alone. The stress due to the pressure P is now P/bd lbs./square inch; this is represented by the line ik parallel with

fg. The total pull in the steel is then equal to the area of the triangle khl times the width b of the section. For slabs or arches the width is usually taken as 12″. The final concrete stress ei must not exceed the allowable stress; we can therefore arrive at a preliminary estimate of the dimensions required by Formula 16, assuming a materially lower "allowable stress" for the concrete, and a higher stress for the steel, when making the first trial.

If the section is one in a *column* the calculations are essentially different. The eccentrically loaded column is of frequent occurrence; in fact, few columns are always loaded centrally. In practical cases it is almost always impossible to calculate the eccentricity of the load, and elaborate formulas are therefore of little or no use. Tension should never occur in the column; if there is tension with the selected arrangement it is better to change the lay-out. The percentage of steel will always be much greater than in the case considered above, and, as there is no tension, we may perhaps calculate our column as a homogeneous section, using, however, for the moment of inertia the expression

$$I = I_c + r\, I_s \tag{27}$$

where $\begin{cases} I_c = \text{mom. of inertia of concrete alone.} \\ I_s = \text{mom. of inertia of steel alone.} \end{cases}$

The cases where the condition of loading can be ascertained with any degree of certainty are very few indeed, and when they do occur the bending moment is likely to be very small. If such is the case it is simpler and probably as correct to calculate the column as a pure column, using a correspondingly higher factor of safety, and then, if necessary, finally investigate the problem assuming the neutral axis to be disposed at the center of the section, and take the moment of inertia with reference to the center line.

82. **Chimneys.** As an example of approximate methods of calculating a piece subject to bending and compression, let us consider a single shell chimney of uniform thickness. The diameter d (in feet) of the flue is given, and so also the height H (in feet). Let the outside diameter be D (in feet); the area presented to the wind pressure (w lbs. per sq. ft.) is then DH

square feet, and the total pressure DHw lbs. Hence the bending moment at the base (the overturning moment) becomes

$$DHw \text{ times } \tfrac{1}{2} H = \tfrac{1}{2} wDH^2 \text{ lbs.} \times \text{feet.}$$

If now the total allowable compressive stress on the concrete is C lbs./sq. in. and the compressive stress due to the weight of a column of concrete 1″ square and H feet high is (approximately) H lbs./sq. in., then the compressive stress due to the overturning moment must not exceed $C - H$ lbs./sq. in. Assuming the neutral axis to go through the center of the section, which indeed is not true, and disregarding further the benefit derived from the steel in the compressive side (which is on the safe side), the moment of inertia of the ring is

$$\frac{\pi}{64}(D^4 - d^4),$$

hence
$$(C - H) \cdot 144 = \frac{\frac{1}{2} \cdot wH^2 \cdot D}{\frac{\pi}{64}(D^4 - d^4)} \cdot \frac{D}{2},$$

which, when solved, gives the outside diameter

$$D = \sqrt{\frac{w}{18\pi} \cdot \frac{H^2}{(C - H)} + \sqrt{\left(\frac{w}{18\pi} \cdot \frac{H^2}{C - H}\right)^3 + d^4}}$$

and the tension per inch of circumference becomes

$$\tfrac{1}{2}(C - 2H)(D - d) \text{ lbs.}$$

83. **Footings.** In Figure 120, $2R$ (inches) denotes the side of the footing, $2r$ (inches) the side of the column. The bending

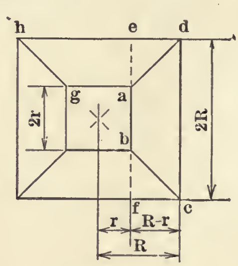


FIGURE 120.

moment on side ab (considering the footing as a cantilever-slab) corresponds to the loaded area $dabc$. We have, for a load p lbs./sq. in.:

Load $dcef = (R - r)\, 2Rp$; arm of bending moment around $ef = \tfrac{1}{2}(R - r)$.

Hence bending moment

$$A = pR\,(R - r)^2;$$

load *aed* plus *bcf* $= (R - r)^2\,p$; arm of bending moment around $ef = \frac{1}{3}(R - r)$.

Hence bending moment

$$B = \tfrac{1}{3}\,p\,(R - r)^3$$

The total bending moment due to the area *abcd* is then the difference between A and B;

$$m = A - B = (R - r)^2 \cdot p \cdot (\tfrac{2}{3}R + \tfrac{1}{3}r)$$

and the depth of footing becomes, according to Formula 8, for a width of beam $2r = b$

$$d = \frac{1}{c_1}\sqrt{\frac{m}{b}} = \frac{R - r}{c_1}\sqrt{\frac{p}{6}\left(\frac{2R}{r} + 1\right)} \qquad (28)$$

to which corresponds a pull s_t in the steel, for the distance *ab*

$$s_t = \frac{c_2}{12} \cdot 2\,r \cdot d = \frac{c_2}{6}\,rd$$

It is, however, quite necessary to provide reinforcement for the portions *ae* and *bf*; for this reason the amount found above may be multiplied by a factor estimated at about 2, which gives:

$$s = \frac{c_2}{3}\,rd \qquad (29)$$

for each layer of steel (Figure 121). The radius of the column

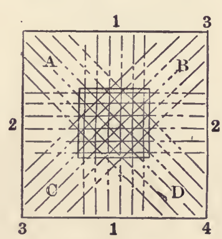


FIGURE 121.

should be made as large as possible, because a material saving in depth of footing is obtained thereby; usually the column must have an enlarged base for other reasons as well. In Articles 14 and 15 we found the cross-sectional area of column:

$X = 1400\,F$ for a hooped column

$X = 1060\,F$ for a plain reinforced column, so that the average pressure, under the conditions assumed, is 1400 and 1060 lbs./sq. in., respectively. With higher percentages of reinforcement these pressures may become materially higher; the column base is therefore enlarged so that the pressure on top of the footing does not exceed the allowable unit pressure, and a steel plate is put under the bars in order to distribute the pressure over the requisite area. According to tests by Bach this allowable pressure may be somewhat increased, owing to the reinforcing effect of the surrounding concrete of the footing, but it does not seem wise to exceed say 1000 lbs. per square inch. The thickness of the plate may be approximately determined by means of a formula by Grashof:

$$t = \frac{r}{200}\sqrt{p} \qquad (30)$$

where

t = thickness of plate, in inches,
r = radius of reinforcement (= $\frac{1}{2}$ of diameter of column, less 2″), in inches,
p = pressure on plate, in lbs. per square inch.

The dimension g in Figure 129 may be found by Formula

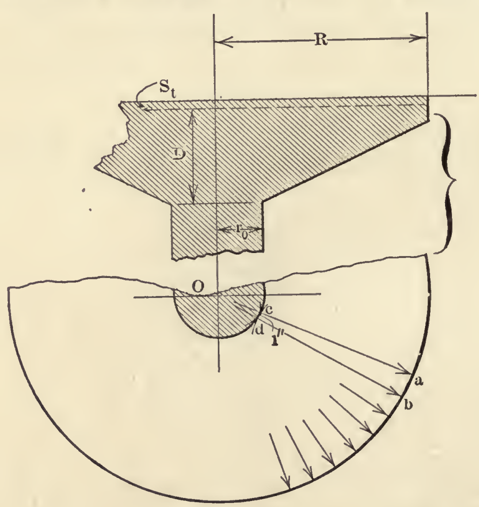


FIGURE 122.

28 above, using for p the allowable pressure on the concrete.

84. **Circular Reinforcement in Plates.** The circular plate in Figure 122 is supported on a central column. The load is

uniformly distributed over its surface, or symmetrically and continuously disposed along the circular circumference. A segment, *Oab*, will then be subject to a certain bending moment, which moment determines the depth D at the circumference of the column. It is now easy to show that when the load is uniformly distributed over the entire surface, the same formula applies in regard to depth as was derived above for a square footing; the calculations are practically the same and need not be repeated here. When the load is distributed along the edge, the Expression 32 in the following article may be used.

In any case, we will assume that the depth is known in the thickest part of the plate (at the edge of the column). If now the distance dc is one inch long, we have by Formula 9

$$s_t = \tfrac{1}{12} c_2 D,$$

which expression leads to the amount of steel required along the radii, the bars being 1″ apart on the circumference of the column. Imagine now that all these radial bars be cut asunder over the top of the column — disregarding the tensile strength of the concrete, each bar will then have a tendency to move outward, so that if a steel ring surrounded the entire plate, each bar would exert a pressure s_t against the inner face of the ring. If now dc equals one inch, then ab equals one inch times R/r_o, hence the pressure on the ring, measured in pounds per lineal inch of circumference, equals $s_t \times r_o/R$. The tension on the ring is then

$$S_R = s_t \cdot \frac{r_o}{R} \cdot R = \tfrac{1}{12} c_2 r_o D. \qquad (31)$$

It is now evident that the ring with radius R and designed to resist the tension S_R is, mathematically, sufficient reinforcement, so that the radial bars may be dispensed with. In actual practice this is somewhat modified owing to the fact that concrete shrinks when setting, so that it would pull away from the ring; the ring would therefore exert no pressure against the concrete until a substantial, and perhaps dangerous, deformation had taken place. But when the ring is used in combination with a radial reinforcement and when at the same time the depth D is not too small compared with the radius, say D larger than $\frac{1}{3} R$, then the ring would seem to be a very efficient reinforcement. Direct proof of this statement is indeed missing, but the "Mushroom" floors furnish at least some indirect information in this respect, as they probably owe their strength in a great measure to the intelligent use of circular reinforcement. That this type of reinforcement is successful in other types of structures may be seen from the remarks made under "columns," where hoops are extensively used to take care of stresses somewhat similar to those existing in a plate, although the plate at the same time acts as a beam. Exact analysis is of course difficult in these structures which border upon the class where reinforcement may sometimes be omitted entirely: it is well known that tapering footings are often constructed without steel, and the same may be true of columns in special cases.

85. Theory of plates. — The "Mushroom" System.

A reinforced concrete floor without beams or girders is first indicated and patented by Mr. C. A. P. Turner of Minneapolis. As far as known there is no perfectly satisfactory way of finding the stresses in constructions of this kind, although buildings actually constructed on this principle have given good satisfaction, according to the published records. The stresses must necessarily be of a very complicated nature, especially under concentrated or unsymmetrical loads; the following analysis does not pretend to solve the problem in anything approaching a general way, and the formulas apply only in case the entire building is loaded with a uniformly distributed load. The formulas are not inconsistent with the assumptions made for reinforced concrete construction, and they are therefore presumably a step in the right direction. It is well known that most of the proposed formulas are based upon the theoretical strength of the plates with equal tensile and compressive resistance, and reinforced concrete does not possess any such qualities.

Figure 123 shows the general scheme for a floor of this kind: the floor slab is simply a flat plate resting upon columns, the tops of which are enlarged. Let the uniformly distributed load be w lbs./sq. foot and the span l feet. The slab is divided into six strips: two diagonal strips AD and BC, and four strips along the sides AB, BD, DC, and AC. If we now suppose the panel to be square, the load on each of the crossing diagonals may be taken as $\frac{1}{2}w$, while the span $AD = BC = l\sqrt{2}$. Then, by Formula 30 for AB: $d = \frac{l}{c_1}\sqrt{\frac{w}{q}}$ and for BC: $d = \frac{l\sqrt{2}}{c_1} \cdot \sqrt{\frac{\frac{1}{2}w}{q}} = \frac{l}{c_1} \cdot \sqrt{\frac{w}{q}}$

so that the depth is uniform, and our problem centers around the design of a side strip like AB. The notations are shown in Figure 124, where

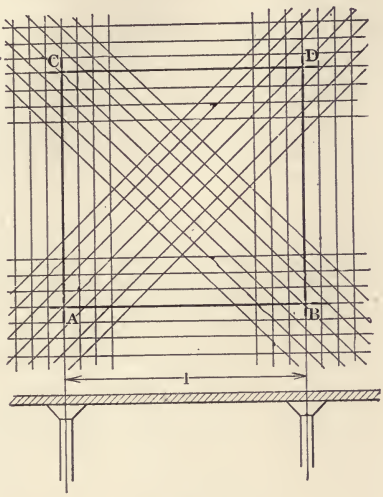


FIGURE 123.

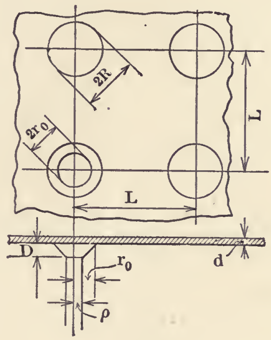


FIGURE 124.

L inches is the span between column centers.
p the load in lbs./sq. inch.
R the radius in inches of a certain circular plate.
r_o the radius in inches of the support under the plate, here referred to as the " cap."
ρ the radius of the column in inches.
d the depth of the slab in inches.
D the depth of the cap in inches.

We will now proceed as follows: We consider the floor slab as supported on the edge of the circular plate with radius R; this plate will then have a uniform load on its surface and a concentrated load along its circumference. Finally the " cap " with radius r_o will be designed for a load concentrated on its circumference, disregarding the uniform load on its surface.

The total area of the floor panel between the four column centers is L^2 square inches; the total weight corresponding to this area is pL^2 lbs. The area of the circular plate with radius R is πR^2, the total weight on same $p\pi R^2$. Hence the weight of the portion outside the circular plate becomes $p(L^2 - \pi R^2)$ lbs.

In the following computations, L^2 is always large compared with πR^2 so that this quantity may be neglected, which is also on the safe side. The load is therefore pL^2, and as the circumference of the circular plate is $2\pi R$, the load per lineal inch of circumference becomes $\frac{pL^2}{2\pi R}$, producing a bending moment equal to $\frac{pL^2}{2\pi R}(R - r_o)$. If measured per lineal inch of the circumference of the cap with radius r_o it becomes, by multiplication with R/r_o (Figure 125),

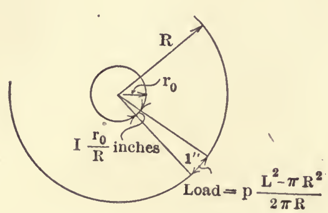


FIGURE 125.

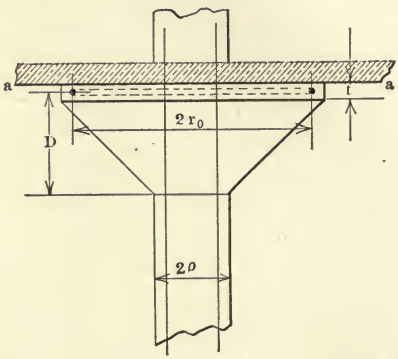


FIGURE 126.

$$m = \frac{pL^2}{2\pi r_o}(R - r_o)$$

and the corresponding depth, for $b = 1''$

$$d = \frac{L}{c_1}\sqrt{\frac{p}{2\pi r_o}(R - r_o)} \qquad (32)$$

for the circular plate. For the slab portion we have, by Formula 16:

$$d = \frac{l}{c_1}\sqrt{\frac{w}{q}} = \frac{L}{c_1}\sqrt{\frac{p}{q}}. \qquad (33)$$

The value of r_o must now be such that the two depths become alike, which gives

$$r_o = \frac{q}{2\pi + q}\cdot R; \qquad (34)$$

at the same time, the value of R is determined by the selected value of q by formula

$$R = \tfrac{1}{2}L\left(1 - \sqrt{\frac{8}{q}}\right),$$

see Article 67, Formula 25. The depth of the cap is found by the formula above, as the load again is pL^2, substituting only r_o for R and ρ for r_o, hence

$$D = \frac{L}{c_1}\sqrt{\frac{p}{2\pi\rho}(r_o - \rho)}. \tag{35}$$

According to Article 84 the reinforcement may be disposed in a ring with radius r_o; the tension in this ring becomes:

$$S_r = \frac{1}{12}\, c_2 r_o D \tag{36}$$

The arrangement is shown in Figure 126, where the thickness t should be about 4″ so as to cover the ring thoroughly. The cap should be cast in one piece with the column, but there is no reason why a joint may not be made between the top of the cap and the bottom of the flat portion along line a—a in Figure 126.

The reinforcement for the flat portion is designed as for any other slab. We have the depth

$$d = \frac{L}{c_1}\sqrt{\frac{p}{q}} \tag{33}$$

and the corresponding pull in the steel S_f tons, for a band one foot wide, is therefore, according to (17)

$$S_f = c_2 d \tag{37}$$

This reinforcement should be disposed near the bottom of the slab at the center of the span, and near the top over the columns. It will be seen that this leaves a considerable space around the column without reinforcement near the bottom, which should be avoided. We may therefore follow the prevailing practice and bend every alternate bar up, leaving the balance of the steel straight near the bottom. The reinforcement over the column is then inadequate, and we will have to introduce additional steel at that point; — if we decide to use rings we may use one ring with radius R, the strength of which is determined according to Article 84 by the formula

$$S_R = \frac{1}{12} R c_2 d \tag{38}$$

We have now (Figure 127)

Thickness of slab, in inches $\qquad d = \frac{L}{c_1}\sqrt{\frac{p}{q}} \qquad (33)$

Pull in slab steel per foot width, tons	$S_f = c_2 d$	(37)
Radius of upper ring, inches	$R = \frac{1}{2} L \left(1 - \sqrt{\frac{8}{q}}\right)$	(25)
Tension in upper ring, tons	$S_R = \frac{1}{12} R c_2 d$	(38)
Radius of cap-ring, inches	$r_o = \frac{q}{2\pi + q} \cdot R$	(34)
Tension in cap-ring	$S_r = \frac{1}{12} c_2 r_o D$	(36)
Depth of cap, inches	$D = \frac{L}{c_1} \sqrt{\frac{p}{2\pi\rho}(r_o - s)}$	(35)
q is the factor of continuity,	$q = 10$ to 16.	

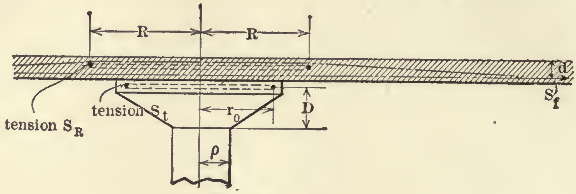


FIGURE 127.

All dimensions are in inches; the load p is in lbs. per square inch and includes both the given live load and the weight of the construction itself.

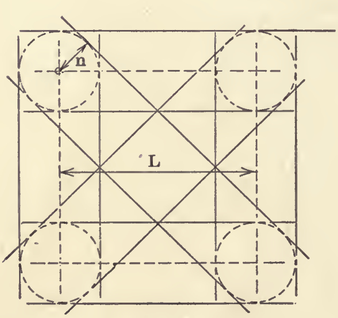


FIGURE 128.

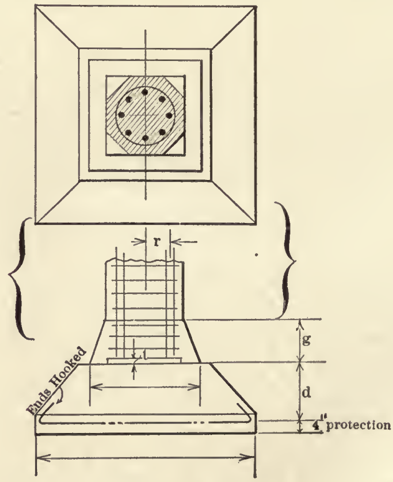


FIGURE 129.

The plan, Figure 128, shows the disposition of the strips. If we draw the four circles with radii n, and if $n = 1/5$ of L, the outlines of the strips will be determined as tangents to the circles, and all portions of the slab will be covered with reinforcement.

It is evident that this entire treatment cannot lay claim to great exactness, as the stresses probably are very much more complicated than here assumed. In any case, the solution here given is correct under the assumption only that the entire floor is covered with the same load at all points. The results are somewhat in accordance with current practice for $q = 16$, so that the tests made on actual structures of this kind may to some extent be taken as circumstantial evidence of the soundness of the formulas, if not of the argument on which they are based.

It must be noted that these formulas do not apply to wall panels, because continuity of construction does not obtain at those points, while also the arrangement is unsymmetrical. The outside bays should therefore always be carried on girders and beams in the usual way; it seems, however, that in some applications of this type of floor the flat construction has been carried entirely out to the walls.

CHAPTER IX

INITIAL AND ALLOWABLE STRESSES

86. CALCULATION of the "initial stresses" in reinforced concrete structures is an impossibility with the data on hand at the present time, hence it becomes impossible to combine these stresses with those considered in the preceding articles, the "static" stresses. What we know about the initial stresses is due chiefly to the careful investigations of Considère; the results may briefly be described thus:—

87. **Concrete Setting in Air Shrinks**, the more so the richer the concrete is in cement. If this shrinkage can take place unrestrainedly no stresses are set up; in reinforced concrete the steel will naturally counteract the shrinkage so that the steel is compressed and the concrete put in tension. This rather benefits the beams, as the tension below the neutral axis is disregarded in any case, while the compression in the steel decreases the tension stresses produced under load. On the compression side, the tensile stresses in concrete due to shrinkage counteract the compression produced by the load, so that on the whole the initial shrinkage stresses are beneficial in beams. In the case of a column this is entirely reversed, as the initial compression in the steel must be added to the compression due to the load, while on the other hand the compression in the concrete is less than calculated. In the hooped column the shrinkage of the concrete has some influence on the stresses in the hoops: — a higher intensity of lateral pressure is required in order to bring tension on the hoops.

88. **Concrete Setting in Water Swells**, the more so the richer the concrete is in cement. In un-reinforced concrete these stresses, if restrained, are beneficial; in reinforced concrete the swelling puts the concrete in compression and the steel in tension. Usually reinforced concrete members set in the air, except footings, sea-walls, and such structures; it must therefore be

possible to keep the concrete in such a moderate state of moisture that no initial stresses of importance are set up during the hardening period. Hence the necessity of sprinkling the concrete freely for the first two or three weeks; this should also be done liberally in order to furnish the setting concrete with the necessary water to make the chemical action in the cement take place as required.

89. **When once the Concrete is Hard and Dry**, addition of water makes the concrete swell; these variations in volume are the more dangerous the less cement the concrete contains, because the stresses produced are nearly the same if not higher, while the resistance against tensile stresses is, of course, less in the leaner concrete. It is important to keep the concrete equally moist all the time, and water should therefore be put on regularly and frequently; on concrete three or four weeks old, and dry, a sudden addition of large quantities of water may prove injurious. If, however, the concrete has been test-loaded so that larger stresses have existed in the concrete prior to the wetting, no change in volume seems to take place.

90. It is a question of great importance to settle the exact intensity of stress in a reinforced concrete member before the load is put on. Only then will it be possible to design concrete structures with absolute economy: the better informed we are in regard to the distribution of stresses in any given case the smaller can we make the factor of safety. It seems indeed that the shrinkage stresses are large enough to crack reinforced concrete slabs, even sometimes beams; the best remedy is to keep the concrete moist, so as to avoid excessive stresses, and to extend the bars well beyond the supports, or otherwise anchoring the bars, to prevent sliding. While cracks certainly look bad, this kind of cracks cannot have any great effect upon the initial stability of the structure, because we assign no tensile resistance to the concrete. There is no definite or conclusive information available to the writer giving data on the durability of members cracked in this way; we are perhaps justified in concluding that no bad action takes place. The only danger seems to be from corrosion of the exposed steel, but the efflorescence comes to our help in this case, frequently filling the cracks completely.

91. Temperature stresses do not usually exist in unrestrained reinforced concrete members, because the concrete is a fairly

good conductor of heat, and the coefficient of expansion is nearly the same for steel and for concrete. In large buildings, temperature expansion may indeed cause some trouble, because the concrete, in expanding, throws the columns out of plumb and causes the walls to turn. Expansion joints are frequently made in long buildings; in later years, however, expansion joints are not used as much as they were, except of course in retaining walls and similar structures where frost and heat have unchecked sway. In all structures such as arches, continuous bridge girders, etc., a serious error may be committed by disregarding the temperature stresses.

92. The expansion joint as used for plain concrete work does not interest us in this connection. In the reinforced concrete wall it is indeed questionable whether more is not lost by giving up the continuity than by having a few fine cracks at intervals. It must be remembered that usually the expansion joint is a point of discontinuity in the steel as well as in the concrete, and a section exposed to accidentally higher loads or stresses than planned loses the support of adjacent sections which perhaps are not quite so overloaded. In any case, an expansion joint must be a clean joint through the entire body of the concrete; simply marking off of the surface does not constitute a joint.

93. **Allowable Stresses.** The analysis given in the preceding articles applies, in so far the mathematics are concerned, to any composite material having properties consistent with the assumptions made. These are, as will be remembered, that (1) the concrete has no tensile resistance, (2) that sections plane before loading remain plane after loading, and (3) that the coefficient of elasticity remains constant up to the point of loading investigated. In addition, a number of assertions have been made, for instance that a bond exists between steel and concrete, that concrete has a lateral expansion, that continuity exists in reinforced concrete beams, and other similar statements. As a matter of fact, the concrete has a definite tensile resistance, the coefficient of elasticity is not a constant as usually determined, and it is doubtful whether or not the sections remain plane. That the bond exists cannot be doubted, and the lateral expansion as well as the continuity are well-established facts. The question here is their numerical value, without which we cannot design consistently and economically.

94. The allowable stresses are used in the design for the purpose of obtaining an ample "factor of safety." At the present time, the relation between allowable stress and factor of safety is in doubt, and it will probably always remain so. The reason for this is that, even with all the materials stored before our eyes and open to investigation, the strength of the concrete cannot be predicted with certainty, much less the bonding strength. Even the strength of a given cement, mixed with a given quantity of water in a room of given temperature, is a variable quantity as reported by different testing laboratories. In addition, the more recent experiments made on reinforced concrete specimens show that their ultimate strength is not an absolute quantity, but depends within wide limits upon the number of repetitions of the load. It follows that a very large number of experiments made in the usual way — loading the specimen once only — are misleading in their results and cannot be of much value unless compared with tests in which a large number of repetitions of the load have taken place.

95. In spite of these apparently unsurmountable difficulties, reinforced concrete design is at present established on a fairly firm basis, especially if compared with design involving the use of other engineering materials. The strength of wood, of natural stone, and even of steel, is subject to doubt in many cases. It is only necessary to point to such questions as those connected with the strength of steel columns with more or less " fixed " ends to show that not everything is settled beyond doubt, and many other instances could be cited. But we have to bear in mind that the allowable stresses assigned to reinforced concrete are principally established through practice and have but little to do with the laboratory experiments. The actual proof of the stability of reinforced concrete construction is furnished by the many splendid structures erected of this material, and only to slight degree by the many haphazard experiments made, out of which most any kind of a theory could be construed. The difference between concrete as actually used and as tested in the laboratory is, that in the building all steel rods are securely anchored in the adjacent span (or ought to be), while in the laboratory individual beams are tested without any arrangement to secure the proper sliding resistance. When these results are analyzed by means of an erroneous shear-theory, it is no wonder

that the "shear" resistance today, after 25 years of experimenting, is as much in dispute as ever. The same applies to bonding tests: the diameter of the concrete specimen is entirely disregarded, yet this dimension is at least as important as the length of embedment.

96. For these reasons, the allowable stresses are not taken as a certain fraction of the ultimate strengths, or, if they were, that fraction would not necessarily be the "factor of safety." The allowable stresses are fixed by practice grown out of the accumulated experience of many years, as a compromise between the conservative designer on one side and the economist on the other side. They have no meaning whatever unless accompanied by an extensive set of specifications calling for certain materials prepared in a certain way, and they are therefore largely a local issue to be determined by the quality of obtainable and prevailing materials in each locality, coupled with and correlated by the obtainable engineering supervision prevailing in that same locality. This supervision is always necessary; not only because the temptation to "save" may be too great, but much more because the *intelligent* and *efficient* handling of the concrete is, in reality, the factor which determines the final factor of safety. It is the duty of the inspector to enforce the specifications in letter and spirit; it is not less the duty of the engineer to draw up specifications which can be enforced, and at the same time compel the use of first-class materials. It must not be understood that good inspection alone will save the reinforced concrete job from all dangers: willing coöperation between contractor and engineer, inclination on the owner's side to pay a fair price for the work, and, most of all, a full understanding of the why's and wherefore's are indispensable.

97. With first-class materials, it is customary in the United States to use the following allowable stresses:—

(*a*) *Columns.* The allowable stress on the steel is dependent upon that used for the concrete; we have $S = rC$. It is usual practice to take $r = 15$ or 20 for columns; the latter figure prevailing. It is common practice to allow 500 lbs./sq. inch for concrete when the columns are centrally loaded; when a small eccentricity exists which cannot be estimated in figures 400 lbs./square inch may be allowed. This is for a concrete mixture having a mortar base of one part of cement to two parts of sand;

according to the size of the column and of the aggregate the proportion of stone may be from two to three times the volume of the cement. However, when first-class materials are used for a first-class job, these stresses are very conservative, and 600 to 700 lbs./square inch are frequently used when the percentage of steel is low. It stands to reason that large amounts of steel should be avoided, the more so the higher the stresses on either steel or concrete.

(*b*) *Floors.* While it is questionable practice to use different mixtures for different parts of the same building, it is frequently done because the richer mixture makes it possible to decrease the size of the columns, while economy calls for as lean a mixture in the floor as may consistently be used, while at the same time the leaner mixture is less liable to cracks. Some engineers use therefore as lean a mixture as 1:3:6, corresponding to an allowable stress of 450 or 500 lbs./sq. inch, while 600 lbs./sq. inch is used for a 1:2½:5 mixture and 700 for a 1:2:4 mixture. It is recommended to use the higher stress and the richer mixture, and avoid the cracking as far as possible by liberal sprinkling; the author does not consider the usual mixture of 1:2:4 as being very good for thin reinforced concrete floors and would prefer 1:2:3½ as allowing a greater latitude in the manipulation. While the steel stress is fixed in its relation to the concrete stress in columns, there is no such relation for the floors. It is customary to use 16,000 lbs./square inch for low-tension steel, and 20,000 lbs./square inch for high-tension steel. At the same time, proper anchorage is provided for by extending the steel bars into the next bay, or by hooking the bars with an open hook at the end; see Article 8. The higher the stress, the longer should be the embedment, hence for low-tension deformed bars 24 diameters should be used, and 36 diameters for high-tension deformed bars. Plain bars should, as said, have an additional hook on the ends equal in length to 6 diameters. It is prevailing custom to make $r = 15$; for continuous construction q is usually taken as 10, while for non-continuous beams 8 is used.

(*c*) *Other Structures.* Arches are usually designed by calculation of the bending moments and thrusts due to the live load, the weight of the structure itself and the fill on same, and the changes in temperature existing in the locality where the arch is to be built. The allowable stresses are then usually taken as for columns; the tension in the steel (if any) should not exceed 10,000 or 12,000 lbs./square inch, but much depends upon the care and method of analysis. Some engineers consider arbitrarily the condition where one-half of the arch is loaded over its entire area; it is by no means certain that this load is the most dangerous. Where the analysis is based upon the actual maximum stresses, the allowable stresses may be somewhat increased, especially where the foundation is of unyielding nature, as bed rock, and the abutments are of sufficient area and weight.

Retaining walls may be considered as pieces subject to bending in case of the modern, ribbed construction. Special attention must be paid to the proper anchoring of the tension reinforcement, as walls of this kind usually are built on the cantilever principle.

Stand-pipes and water pipes subject to internal pressure require low stresses in order to become water-tight under pressure; 10,000 lbs./square inch in tension on steel should be the upper limit. The stability of the finished structure depends upon the integrity of the joints between the hoops belonging to the same circle or spiral, hence the thickness of the concrete and the length of "lap" are most important factors. The concrete should be at least equal in thickness to 10 diameters of the embedded steel bars if these are comparatively heavy and spaced comparatively far apart, say 8 to 12 inches; if light, closely spaced bars are used the thickness should be increased rather than decreased, and special pains taken with each joint. The best way is to break joints whenever possible so that no two adjacent joints come in the same plane, to keep the bars to be joined a distance apart equal to two steel diameters, and to surround the joint for its entire length with a coil. It is not often that all these precautions are taken at once. It has been found, however, that stand-pipes may fail by the concrete inside the hoops separating from the concrete outside the hoops, thus destroying whatever bond may have existed between the steel and the concrete. This danger is best avoided by having no vertical, concentric planes of weakness in the concrete.

98. The factor of safety of a reinforced concrete structure depends therefore upon the selected stresses for the two materials and the bond stress selected for anchoring the ends of the bars; also upon the selected value of the ratio r between the coefficients of elasticity of the two materials, and finally upon the degree q

of continuity. All of these influence the safety of the building, each in a different manner, and some in a different manner at different times. It has therefore been found impossible to establish a definite "factor of safety" for reinforced concrete buildings, but we know that the carrying capacity of a building designed as here described, and erected in a first-class manner, will easily carry three times the calculated dead and live load under one or a few test loads. We also know that it will carry a very great number of repetitions of twice the calculated total load, but it will probably not carry an unlimited number of repetitions of three times the calculated total load. It follows that a material increase in the allowable stresses suggested is dangerous at the present time.

PART III

PRACTICAL CONSTRUCTION OF REINFORCED CONCRETE BUILDINGS

BY ERNEST L. RANSOME AND ALEXIS SAURBREY

CHAPTER X

MATERIALS OF CONSTRUCTION

Requirements and Tests

Cement. The strength of concrete depends principally upon the quantity and quality of cement used. In order to insure a satisfactory and uniform grade of cement, the shipments are tested according to rules laid down by the "American Society of Civil Engineers," and the results must conform to the requirements of the standard specifications prepared by the "American Society for Testing Materials." Copies of these publications may be obtained from the societies named, but, as the rules are subject to variation, the specifications are not printed here.

It is an open question to what extent test reports may be depended upon. The specifications call for certain numerical values to be obtained, but it is now well established that no two individuals can obtain the same numerical result from identical samples of cement submitted to them. The reasons for this are many; for instance, atmospheric conditions may be different and many other uncontrollable factors may and do enter. The greatest trouble is, however, that the manipulation influences the strength of the test piece, and no two experimenters will handle the cement mortar in identical manner.

Whatever the reasons — the facts remain. On the following page, Table A gives the results of cement reports made in various laboratories on identical samples of cement: Case I is a cement which, while somewhat deficient in some respects, was nevertheless of fair quality, yet proved to be extremely quick setting on the job. The letters A, B, C, etc., refer to the parties making the test; A, B, and E are testing laboratories in Cleveland, Ohio, all of good reputation; C is instructor in cement testing at a large engineering college; D is a concern of national reputation. The reader is asked to compare the results in each group and then draw his own conclusions. Table B gives the results

TABLE A, SHOWING VARIOUS TESTS ON CEMENT BY DIFFERENT LABORATORIES

					Lbs. per sq. in.					Volume Am. Soc. C. E.			
	Set		Fineness		Strength, neat			Strength, 1:3				Accelerated	
Tested by	Initial.	Final.	100 m.	200 m.	24 hrs.	7 d.	28 d.	7 d.	28 d.	Air.	Cold water.	Steam.	Boil.
Standard	+0.30	1.0 to 10.0	92	75	175	500	600	175	250	—	—	—	—
CASE I:													
A	3.30	6.0	94.4	82.6	165	324	653	159	307	OK	OK	OK	—
CASE II:													
A	3.0	5.0	96.6	85.8	371	679	771	274	351	OK	OK	OK	—
B	1.55	3.0	93.5	82.8	179	—	—	—	—	—	—	OK	—
C	1.35	4.30	97.5	86.0	273	613	—	181	—	—	—	OK	OK
CASE III:													
A	4.45	8.10	94.9	81.7	207	644	749	195	293	OK	OK	OK	—
B	0.45	1.50	93.0	81.5	168	492	585	134	391	—	—	Soft	—
CASE IV:													
A	3.50	5.10	94.6	82.0	267	602	697	252	333	OK	OK	OK	—
C	1.15	3.10	96.0	82.0	273	641	—	190	—	—	—	OK	OK
CASE V:													
A	3.15	5.25	93.7	81.4	233	614	427	261	370	OK	OK	OK	—
C	1.40	4.35	98.0	80.0	117	507	—	121	—	—	—	OK	OK
CASE VI:													
A	4.10	6.25	95.3	82.4	231	602	—	252	—	—	—	Soft Warped Cracked	
C	2.25	4.55	96.2	82.0	90	587	—	150	—	—	—	OK	OK
D	4.30	7.30	94.6	79.2	359	833	—	259	—	OK	OK	Distorted Soft	
CASE VII:													
A	3.35	6.5	93.0	77.2	343	661	—	243	—	—	—	OK	—
C	1.5	6.20	93.7	76.4	179	601	—	154	—	—	—	OK	OK
CASE VIII:													
A	3.20	6.0	95.2	81.2	325	707	799	315	436	OK	OK	OK	—
C	2.15	5.20	92.5	75.5	233	708	—	267	—	—	—	OK	OK
CASE IX:													
A	3.30	6.25	93.4	84.8	363	677	817	383	446	OK	OK	OK	—
B	1.55	4.0	94.9	79.8	203	519	686	212	477	OK	OK	OK	—
CASE X:													
A	3.30	5.30	94.2	83.8	420	670	687	363	428	OK	OK	OK	—
B	2.15	4.35	95.1	80.4	226	539	696	220	483	OK	OK	OK	OK
CASE XI:													
B	2.20	4.45	95.0	80.0	208	507	—	218	—	—	—	OK	OK
B	2.35	4.50	94.7	80.3	202	485	597	207	475	OK	OK	OK	OK
B	2.15	5.10	95.0	79.7	213	495	603	207	485	OK	OK	OK	OK
CASE XII:													
Mill Report ..	3.0	5.58	—	79	290	627	—	224	—	—	OK	—	OK
B	2.25	4.55	92.7	75.1	211	494	611	221	484	OK	OK	OK	OK
B	2.15	5.5	93.1	77.0	—	476	607	215	484	OK	OK	OK	OK
B	2.10	5.25	92.9	76.5	207	493	612	223	478	OK	OK	OK	OK
A	2.30	4.50	94.4	81.4	365	694	699	284	345	OK	OK	OK	—
E	2.43	5.08	95.0	77.9	421	802	846	325	389	OK	OK	OK	OK
CASE XIII:													
Mill Report ..	3.3	6.0	—	79.0	284	690	—	255	—	—	OK	—	OK
A	2.50	4.30	95.2	82.5	429	748	710	302	343	OK	OK	OK	—
B	1.55	4.25	94.9	75.4	—	482	608	177	416	OK	OK	OK	OK
B	2.15	5.0	94.0	77.0	195	482	609	225	479	OK	OK	OK	OK
A	2.30	4.50	94.8	81.8	325	691	696	272	380	OK	OK	OK	—
E	2.43	5.5	96.5	77.5	404	786	877	306	417	OK	OK	OK	OK

The first line of this table shows the requirements of the American Society for Testing Materials in force when this was printed, in December, 1909.

TABLE B

Table Showing Time of Setting of Sixteen Samples from One Car Load

Bag No.	Accelerated Pat Test	Time of set Initial, hrs.	min.	Final, hrs.	min.
1	Hard, sound	1	0	3	5
2	Soft, left glass, slightly warped	3	5	4	0
3	Hard, sound	1	10	3	20
4	Hard, sound	0	45	2	30
5	Hard, sound	0	45	2	45
6	Hard, sound	1	0	3	5
7	Hard, broke glass, slightly warped	1	5	2	50
8	Hard, sound	1	45	3	0
9	Hard, sound	0	45	2	50
10	Hard, left glass	1	0	3	15
11	Hard, glass broke	1	0	3	0
12	Hard, glass cracked	1	5	3	0
13	Hard, sound	1	10	2	55
14	Hard, glass cracked	1	0	2	50
15	Hard, sound	1	50	2	35
16	Hard, left glass	0	55	2	45

Tables from a paper in the Eng. News, Dec. 9, 1909, by Alexis Saurbrey: "Comparison of Reports on Tests of the Same Cement by Various Laboratories."

of simultaneous tests on 16 samples taken from the same car, and show either that the tester was incompetent, which is hardly to be believed, or else that the shipment varied very considerably from bag to bag, which of course was emphatically denied by the manufacturer.

Undoubtedly, there is a vast amount of dissatisfaction with present methods of cement testing. This is not the place to discuss whether the tests now in common use are too difficult to make correctly, or the present staff engaged in cement testing is deficient in skill, or both. Professor Waterbury is the author of the following statement:

"(1) It is nearly impossible for two persons to obtain the same numerical results for tests upon a given sample of cement. (2) The results obtained by any one person, who has had some experience in testing cement, are generally in accordance with other results obtained by the same observer. (3) There is likely to be a greater variation in the results of the 24-hour neat tensile tests than in the result of neat tensile tests for longer periods of time. (4) With the exception of the 24-hour neat tests, there is likely to be a greater variation in the results of 1:3 mortar tests than in the results of neat tensile tests."

Mr. W. P. Taylor, one of the leading cement experts in this country, when addressing the National Association of Cement Users, said in part:

> "Cement testing is a difficult process requiring experience, care, precision, and knowledge, and hence should only be entrusted to well qualified men, but too often this important work is relegated to utterly untrained and careless operators and the results obtained by such methods are really worse than nothing, as they often are positively misleading. Many tests made at the present time by supposedly responsible parties are ridiculous in their inaccuracy, as any one having knowledge of this subject will admit. Instances might be cited without number. In one case a cement was rejected as being quick setting, but an investigation showed that the test had been made in a hot room in a temperature of over 80° F. and the specimen besides placed directly over a radiator — the cement itself was entirely normal. Strength tests are often made by inexperienced boys committing every possible error of manipulation. In one case a cement reported as breaking at 125 lbs. was found to give a strength of over 250 lbs. when accurately tested. Cases of unjust rejection on the accelerated test for soundness through improper manipulation and interpretation of the results are by no means uncommon. Of the sieves used for testing fineness, not one in four has been properly standardized. These inaccuracies, it must be remembered, are not only found in the small laboratories, but only too often in those of some reputation, and the cause may be found to be entirely due to the desire to cheapen the cost.
>
> It should be recognized at once that if cement tests are made it is worth while to make them well, even at possibly a somewhat increased expense."[1]

Granting, however, that satisfactory and reliable cement has been received, it becomes necessary to so store and use the cement that it will not deteriorate. For this reason cement must be stored in a house of substantial design, where water or even dampness will not penetrate. The temperature must be kept as low as possible in the summer, as a temperature of from 80° to 100° may seriously interfere with the setting qualities of the cement, changing normal cement into extremely quick setting cement. This knowledge should always be imparted to every one on the job, so that close watch is kept of all batches deposited in the forms. If the concrete hardens in the wheelbarrows, it must not be used; it is playing with fate to retemper such con-

[1] Quoted from an editorial in the Engineering News, Dec. 9, 1909.

crete with more water, as it hardens very slowly and probably never reaches the calculated strength. Without doubt, many accidents may be traced back to neglect of this one point.

On a certain large foundation job this very thing happened. When the pier forms were removed the concrete was quite wet and soft, and fell entirely apart. Samples were taken, and owing to the very plastic condition of the concrete it was possible to mold test cubes which were allowed to harden. As expected, the 7 and 28 day specimens had barely cohesion enough to stand the handling of placing them in the machine. One wall, 16 inches thick and 4 feet high, was allowed to remain in place, as it carried no loads. When six weeks old it was still so soft that impressions were readily made with the thumb alone, but in the course of a few months the concrete seemed to get quite hard, and it was decided to leave the wall in place. At the present time, the wall is about five years old and apparently in a satisfactory condition. Similar cases have come under observation from time to time, so that the conditions just described are by no means exceptional.

Dampness is best avoided by careful attention to ventilation on clear and dry days. While opinions are divided on this subject, it seems the best, and certainly the safest, practice to reject cement with lumps or cakes.

Under any circumstances, when cement is received it should be well seasoned and ready for immediate use, and it should be used at once. Hence, "warehouse cement" must always be regarded with suspicion. On large work, the most careful cement inspection and the most scientific testing may easily be had at a negligible cost, but on very small work the problem becomes quite difficult. The parties most likely to suffer are the small manufacturer of cement blocks and the sidewalk man.

The cement used in reinforced concrete work is always Portland Cement; the exceptions are so few that they may be said not to exist. Each car load received is sampled individually, a small amount from each bag out of every 30 bags received, or, in case of delivery in barrels, 1 barrel out of every 10 is customarily sampled. These samples are mixed to an average sample representing the car load, taken to the testing laboratory, and there submitted to the standard tests. In order that the average sample may fairly represent the car load, the individual samples from the several bags or barrels must be taken from various parts of the car. The tester marks each and every package with the name of the testing laboratory and the number of the test, each car load receiving its own "test number." The aggregate amount of cement taken out of each car load is about 16 lbs., one-half of which is sent to the laboratory, and the other half is stored for reference by the engineer. Eight pounds are usually sufficient for the purpose of the tests in common use.

On work of any importance it is good practice to make field tests of the cement (setting time and soundness principally) to guard against changes, and also compression tests on cubes made from concrete taken from the mixer or the wheelbarrows. These tests simply supplement the laboratory tests and cannot replace them; however, the compression test on the concrete as actually used is in itself a very excellent check on the efficiency of the mixture used, and gives also important information as to the proper time for removal of the forms. As a matter of fact, the engineer is not greatly interested in the strength of cement as tested in the laboratory under conditions which never obtain in the field, and he is probably relying upon the complicated and difficult laboratory test because nothing better is available. The compression test in the field might well be used more extensively, although, as far as rejection or acceptance of a given cement shipment is concerned, it is of no importance whatever.

The consulting engineer is sometimes called upon to examine an existing structure, and inasmuch as such examination usually has its cause in troubles of various kinds, he might be interested in knowing what the original proportions of the concrete were. Unfortunately, it seems that there is no very satisfactory method whereby such information can be obtained, and if obtainable, testimony along these lines would probably have little weight in court unless a large number of samples were analyzed.[1] This is also true of compression tests made on

[1] See also paper in Eng. News, 1908, p. 46, by Prof. R. L. Walls, who made a successful analysis of this kind.

One case of this kind happened in Oakland, Cal., where skimping of the cement was proved to the satisfaction of the jury by careful chemical analysis, based upon the amount of lime in the analyzed concrete. Many years afterward, I came across evidence of other nature that showed the chemical analysis to be correct. — Ernest L. Ransome.

cubes or cylinders cut from the concrete, because it would be difficult, but not impossible, to show that the pieces were not injured in the process of cutting.

Perhaps it is well here to call attention to the fact that the size and shape of the test specimen has some influence upon its strength, so that results obtained from 4″ and 6″ cubes cannot be directly compared without reference to the laws governing such cases.

In tests on concrete and mortar, the relative size of the aggregate and the test specimen might also have some influence.

Sand. Next to the cement, the sand is the important factor in determining the strength of the concrete. Various elaborate theories exist whereby the proper composition of the sand may be determined; where a large concrete job is to be supplied from a uniform, local supply of large capacity such investigations may, of course, be of great value, as it is possible to determine just what should be added or deducted in order to get the best results. But ordinarily such investigations are of little value, as the character of the sand may vary from day to day, if indeed it does not vary from shovelful to shovelful. Hence a quick and cheap test is required for daily or occasional use, and such a test we have. It consists in simply comparing the strength of briquettes made from "standard" sand and cement with that of similar briquettes made at the same time from the proposed sand and the same cement. Obviously, this method of testing is free from nearly all the objections made against the usual cement test, as only comparison is wanted and not absolute figures. The standard sand is not particularly strong sand if used for building purposes, so that for good results the proposed sand should give a tensile strength 25 to 50 per cent. in excess of that obtained with standard sand; when the strength is about equal, the sand may be termed "passable" if only low stresses are used in the design, while sand well below standard may be rejected without error. Specifications drawn along this line avoid entirely all questionable and unfair regulations. Where water-tight work is required, the more elaborate granularmetric analysis may be used if the supply is uniform in character. Frequently, an addition of a small amount of fine sand, or preferably stone dust, greatly improves the strength of the concrete.

It is well understood by skilled concrete men that the best grade of sand is clean, sharp, and well graded from fine to coarse, and these words are therefore usually inserted in the specifications. It is believed that only in exceptional cases such specifications are enforced, and the policy of writing specifications which nobody can or will enforce is not to be recommended for obvious reasons. Up to three per cent. of impurities are not usually injurious, but sometimes even a much smaller quantity of clay is detrimental, especially if the several grains are covered with a thin film of clay. The test recommended above settles such questions at once, provided that the mixture made in the laboratory is not such that the film is removed; too much mixing is as bad as not enough. On the job, however, the mixing must of course be greatly prolonged if the grains are coated so as to wash the film off the sand.

The specifications must state the maximum size of grain allowed; usually the sand is required to pass a screen with four meshes per lineal inch.

Stone. The stone should be clean and hard, two requirements easily complied with. Loose dust should not be allowed when the concrete mixture is specified in proportions as 1:2:4 or similar ratios, because the dust acts as so much sand and decreases the strength of the mortar base. Some dust always clings to the stone, hence the word "loose" should be used. On the other side, if it should be found desirable to use "run of crusher" with some additional sand there is no reason why good results cannot be obtained in that way with continuous and intelligent supervision. As a general thing, the engineer will save himself a large amount of trouble by insisting upon separate stock piles for sand and stone, and specify his materials by definite proportions. There can then be no room for dispute.

If the specification suggested above is used, the stone should be required to pass a ring $\frac{3}{4}''$ in diameter for thin reinforced concrete pieces as used for floors, beams, and columns; for heavy work, the size may go up to $2''$ ring or larger. The stone should be retained on the $\frac{1}{4}''$ mesh screen, perhaps with a small allowance, so that, for instance, 5 or 10 per cent. may pass through the screen, the balance to be retained. Certain kinds of rock give oblong stones, and a maximum length should be specified, for instance $1\frac{1}{2}''$.

Attention is called to the ever-increasing use of furnace slag, a by-product from the manufacture of pig-iron. Slag makes excellent concrete if used with the proper proportions of mortar, for instance, 1 : 2 : 3 or 1 : 2 : 3½. The slag is very dry and absorbs water in large quantities; the stock pile should therefore be kept soaking wet at all times. Otherwise the concrete may not set up well.

Boiler cinders should not be allowed in reinforced concrete work, as little as soft limestone or soft sandstone, or any kind of stone disintegrating under the influence of the atmosphere. Fair concrete may be made from soft or friable aggregate by limiting the time of mixing to a minimum; *good* limestone makes excellent and very hard concrete, and crushed brick if not very soft makes a very fair concrete for many purposes. Brick dust is a good substitute for natural sand.

Certain kinds of shale have great toughness when in the natural deposit, but fall to a powder when exposed to the influence of the air. Conglomerate cemented together from a large number of small pieces must be prohibited, even if apparently hard. The authors recall one or two instances where this kind of stone led to very serious trouble. Certain kinds of slag contain very injurious chemicals, such as sulphate of lime, etc. Chemical analysis should be insisted upon before the use of an unknown slag.

Steel. The selection of the steel is rather embarrassing, each "system" claiming special advantages of its own. In most cases these advantages exist largely on paper only, the fact being that almost any kind of steel may be used with success. The form of the particular bar proposed is a comparatively small matter; the real importance is in the material from which the bar is manufactured, and the method of manufacture. In their own work, the authors prefer the cold twisted square bar. The specifications should call for minimum ultimate strength, minimum limit of elasticity, minimum percentage of elongation, and a bending test. The engineer is interested in the kind of steel furnished, not in the method of manufacture.

Plain Bars. Round, square, or flat bars are used, but the round bar should be favored as easier to handle, and flat bars are generally considered as giving smaller bonding strength in the concrete. High tension or low tension bars may be used if a proper length of anchoring is had in each case. The high tension bars are frequently rerolled from old railroad rails — in itself a very good idea. But this steel is rather high in carbon and requires extra care in manufacture; rerolling at too high or too low temperatures may be the cause of brittleness and other trouble. Many engineers decline to use rerolled or hot twisted bars for this reason, and it cannot be denied that unless properly tested, such steel may not be what is expected and required. Figure 130 shows a bad piece of rerolled steel.

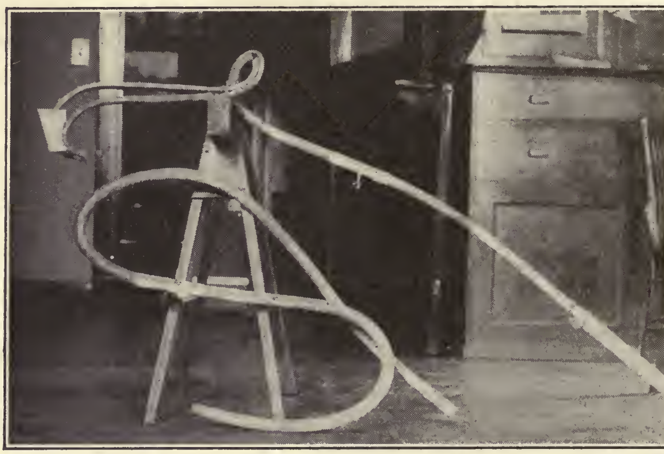


FIGURE 130. UPPER BAR: REROLLED STEEL, IMPROPERLY MANUFACTURED. LOWER BAR: GOOD SPECIMEN.

Both bars from the collapse of the Henke Building (Column Rods).

Photo by Alexis Saurbrey, who examined ruins for the owner.

Deformed Bars. The great class of bars distinguished by projections and recesses on the surfaces have this in common, that if sufficient concrete surrounds the bar, it is harder to pull out than the plain bar of same cross-section. It is almost impossible to discriminate between all these bars which belong to types merging one in the other by gradual steps.

Patent royalties are collected on most of these bars; the twisted bars, both hot and cold twisted, are exceptions. The value of the steel as reinforcement is not greatly affected by the various forms of projections in use; bars with deep grooves should not be used, as water instead of concrete is likely to collect in the pockets, especially on the under side of the bar. If deformed bars are used it should be of a type having the same cross-sectional area at all points of the length.

Wire Mesh. Reinforcement with ready-made wire mesh is adapted only for short span slabs, and even then additional bars of larger diameter are often used. The cost is high, and attempts are therefore frequently made of talking the buyer into allowing much higher stresses on wire fabrics than on plain steel. It is of course possible that our present methods of calculating stresses in slabs are in error, but the proof has yet to be furnished. In the meantime we cannot consistently allow stresses on drawn wire as high as 30,000 or 40,000 lbs./square inch, even if this material has a tensile strength of 100,000 to 120,000 lbs./square inch.

Requirements and Tests for Steel. The ultimate strength of bars used for reinforcing purposes should be at least four times the allowable stress, and the elastic limit should be at least twice the allowable stress. Of these two, the latter is by far the most important; a high elastic limit increases the factor of safety of the entire structure, although not in direct ratio. Both of these figures are easily determined by a tensile test, except in the case of bars without a definite elastic limit, such as cold twisted bars. In this case, the strength and elastic limit may be determined either before or after twisting; generally speaking, the ultimate strength is raised about 33 per cent. by twisting soft stock, while the elastic limit, if it can be at all determined, will be found to be about 75 per cent. of the ultimate strength. This applies to cold twisted bars only; hot twisting does not change the strength or elastic properties of the bar.

The bending test is very important, as practically all the bars are bent on the job; all bending should of course be done cold. For high-carbon steel it is usually specified that the bar must bend cold around a pin four times the diameter of the bar without showing signs of distress. Good cold twisted bars will easily bend around a pin twice the diameter of the bar. Three or four diameters are however more commonly specified. Soft stock should fold flat upon itself without showing signs of cracking or checking. It follows that the kind of steel to specify may depend largely upon the bending problems encountered.

The elongation serves practically the same purpose as the bending test. The minimum amount specified varies from 10 per cent. for high-tension steel to 20 per cent. or 25 per cent. for soft steel. These figures must not lead us to believe that the smaller per cent. of elongation at fracture is a point in favor of the high-tension steel; in fact, the coefficient of elasticity is practically the same for all kinds of steel in common use, and the elongation under a working load depends upon the coefficient of elasticity.

The following quotation from the Cleveland Building Code may be of interest:

"Steel reinforcement shall be divided into two classes, Medium and High Tension. Medium steel shall have an ultimate strength of 60,000 to 70,000 lbs./square inch, and shall conform to the Manufacturers' Standard Specifications as revised Feb. 6, 1903. High-tension steel shall have an ultimate strength of not less than 80,000 lbs./square inch, and an elastic limit of not less than 45,000 lbs./square inch. The elongation shall be at least ten per cent. in eight inches. Bars shall bend cold around a pin of diameter equal to 4 times the least dimension of the bar without showing signs of cracking."

Tiles. The tiles are usually made $12'' \times 12''$ in plan; the width usually cannot be changed, but tiles $12'' \times 6''$ or $12'' \times 18''$ may sometimes be obtained. The thickness of walls and webs is frequently about $\frac{1}{2}''$, subject to variation. Dense or semi-porous tiles can be obtained; it makes little difference in the results which is selected. The surface should be deeply scored so that the plaster may be firmly bound to the tile; only in the roughest kind of work are the tiles left uncovered on the exposed bottom side. In burning, the tiles shrink; a well-burned tile is often as much as $\frac{3}{8}''$ smaller each way than specified. This should be made up in concrete, so that the plans should show the full thickness of floor, not the thickness of concrete topping. At the same time, tiles may be too large, in which case the minimum amount of concrete to be placed over the top of the tiles should be specified. Broken, badly warped, or otherwise defective tiles should not be allowed. Before the concrete is run, the tiles should be made soaking wet, as they will otherwise absorb the moisture from the concrete.

Concrete. Hand-mixing is used only in exceptional cases; the engineer should reserve the right to permit hand-mixing if practically unavoidable. Machine-mixing on continuous mixers is not desirable; the mixer should preferably be a revolving batch-mixer of approved design. The aggregate and cement is measured by volume; it is very convenient to take the cubic foot as unit and consider the bag as containing one cubic foot of loose cement. When the job is started the wheelbarrows or receiving bins are checked by means of the standard unit, and the required depth of filling marked. All wheelbarrows and bins must be brought to a level when filled; a small top on the wheelbarrow load looks like a small matter, but may in fact mean a material decrease in the proportional amount of cement. Wheelbarrows containing the required amount when level full can easily be obtained.

Sufficient water should be added so as to make the mixed concrete into a flowing paste that will pour from the wheelbarrow. The concrete is mixed with an excess of water if pools are immediately formed on top of the concrete when deposited in the forms: the pools increase in size, the water finds an outlet to a lower point, and the cement is washed away from the mortar. Inclined "sand-streaks" on the sides of girders or beams are usually due to this cause. Years ago "dry" concrete was specified, but the manipulation becomes so difficult with dry concrete that "wet" concrete soon became universally used, and at present there is a tendency to exaggerate the amount of water. Where the concreting of large girders proceeds from one end, and the working face of the concrete body is sloping, the excess of water naturally seeks the lower level in the part of the girder box not yet concreted. The water carries "laitance" with it, and this sets without hardening, forming a white plaster-like film on the bottom of the girder, often $\frac{1}{2}''$ thick or more. Such conditions should of course be avoided.

The mixing must continue until all parts are thoroughly incorporated in the mixture and all stones covered with mortar: the concrete will then be of uniform consistency and color, and if sufficient water only is used there will be no precipitation in the bin or in the wheelbarrows of the heavier particles. This separation is much more likely to take place if perfectly clean sand is used, especially if the grains are round, and lake or river sand does therefore require less water and more care than bank sand containing a slight amount of clay. As the aggregate contains varying amounts of moisture in the different parts of the stock-pile, and as the stock pile seldom is perfectly uniform throughout, no hard and fast rule can be laid down for the requisite amount of water. The amount of water used, and the number of turns given the mixer before dumping should be left to a competent man in charge of the mixer. The mixer-man is an important person on the job.

The concrete must be deposited in the forms before it stiffens perceptibly; concrete held longer than that time in either the receiving hopper or the wheelbarrows should be wasted. The concrete must be well spaded and churned in the forms to insure dense concrete and fine surfaces. The entire concrete work should be under the personal and direct supervision of an intelligent and careful man; this man should be present at all times when concreting is going on, and should have no other duties during concreting. If the engineer or owner desires to have an inspector of his own on the job at the same time, so much better, but it should be made clear to the contractor and his men that such inspection is for the purpose of enforcing good work only, not for the purpose of waiving the specifications. The concrete must be conveyed to and deposited in the forms in such a manner that the steel reinforcement is not disturbed and so that older concrete is not injured. The mechanical plant must not be braced to the form work or the building, or have any solid connection therewith. All these rules are dictated by common sense, but nevertheless daily violated, and it is therefore well to put them in the specifications.

Joints in the concrete work should not be allowed except at the natural end of each day's run; it is therefore essential that the plant be in such shape that breakdowns are avoided. The necessary joints must be well-defined, straight lines, preferably through the center of the span of all slabs and beams. The joint should be made perpendicular, not sloping; the vertical joint is easily repaired in case of trouble. New concrete should be joined to old concrete only after the surface of the old concrete has been removed by mechanical or chemical means, and the rough surface made in this way thoroughly cleaned with scrubbing-brushes and water. Neat cement paste is then rubbed into the clean surface, and concreting proceeded with at once. The care taken is of no avail if the cement paste is allowed to dry out or set before the new concrete is put on. The slab and the beam supporting it are usually run in one continuous operation, without any joint between them; but the design can easily be so arranged that a joint can be made if desired.

The concrete gang, and especially its foreman, should be made to understand that it is their duty to make concrete which will not require after treatment to make up for their carelessness or haste. No pointing-up should be allowed before the engineer has seen and approved the concrete; but when so directed the contractor must at once proceed with the pointing.

In monolithic construction, the columns should be run a sufficient length of time ahead of the floor, to allow the concrete in the columns to settle and shrink; the interval may conveniently be utilized in putting the floor-steel in place.

The setting of the concrete is greatly influenced by atmospheric conditions. Hot weather accelerates the action, and cold weather retards it. Otherwise, neither heat nor cold need have any injurious action on the concrete if proper precautions are taken. In hot weather, it may be necessary to cover the green concrete against the direct rays of the sun, and in any case the concrete should be sprinkled liberally to make up for the loss of water by evaporation, as concrete cannot gain its full strength without water. Much more serious is the action of frost, and especially of repeated freezing and thawing; the precautions to be taken in the summer are simple and cheap compared with those required in winter, where the weather may suddenly change from mild to bitter cold. The concrete is made with heated materials and heated water; the green concrete is covered with a tent or boards with straw on top, but not manure, which is said to injure the concrete; in fact, one or perhaps two accidents have been ascribed to the use of manure. Moist heat is supplied to the space below the green concrete, and between the concrete and the covering.

CHAPTER XI

FLOOR SYSTEMS

Broadly speaking, reinforced concrete may be used in one of two ways: cast in place, or cast in the yard and erected afterwards when hard. In the first case, the construction becomes more or less continuous by virtue of the method of erection, and the continuity is then usually emphasized in the design, so that all parts are thoroughly tied and united together. This type of construction is therefore referred to as "Monolithic." In the second case, the structure is divided into separate pieces or "units," and both the designer and the erector must then so articulate the building that the weights and dimensions of the several pieces come within reasonable limits. In distinction from the monolithic work, this type of construction is referred to as "Unit." A large number of "systems" exist within either of these broad divisions, several of which claim protection under United States patents, and all of which claim either superior strength or greater economy. It would, however, be outside the scope of this book to enter into a discussion of these points; moreover, there is no accepted standard of comparison, so that, in all probability, the contractor's bid or proposal on any given building gives the only safe way of determining the relative cost in each case.

Some types of construction are essentially monolithic, such as the flat-plate and column construction, and the tile concrete construction. Others are essentially of the unit type, such as the Visintini System, where each unit is a small truss in itself. The common "ribbed floor," with beams and girders supporting the floor plate proper, is usually built in a strictly monolithic way, but recently considerable efforts have been made toward perfecting the "unit" method for floors of this kind. Two distinct methods have been followed in the unit construction: (1) Each beam or girder is a complete carrying member in itself, and in that case the slab portion is usually molded on the ground, and set in place when hard. (2) The beams and girders are not complete carrying members, the floor slab proper forming the upper flange of the T-beam, and in this case the beams, girders, and columns are cast on the ground and set in place when hard, while the slab proper is cast in place over the top of the beams, and serves the dual purpose of tying the building together, and of forming the compression flange of the beams and girders.[1]

Design. After the mathematical design has been perfected, additional steel must be introduced to take care of shrinkage stresses, especially in the slabs. These bars are disposed crosswise over the tension rods, and also serve as "*distributing rods*"; the structures erected under the so-called "Monier System" always had large quantities of such bars which greatly strengthen the building as a whole. The exterior belt courses should have ample additional reinforcement, and these bars should run continuously around the entire building at each floor, with sufficient lap at each joint. In the "tile-concrete" construction, the value of such additional steel is too frequently overlooked, although it is here of particular importance owing to the absence of secondary beams. It seems to be an open question to what extent the slab can be considered as active in compression, and the laws governing the influence of the width of the top flange are practically unknown except for a few sporadic tests. It can, however, be stated that the active width of slab depends upon the thickness of the slab, and upon the intensity of compression stress, and the stiffness of the system as a totality probably enters to some degree. Common rules are: One-third or one-sixth of the span; two-thirds of the distance between beams; six or ten times the width of the stem, etc. None of these rules is derived from either test or convincing analysis.

In monolithic construction, the questions of bearing for beam — or girder — ends do not usually arise, and when they do, the strength of the supporting material is usually the governing factor. In many brick buildings the pressure on the wall bearing has been limited to 200 or 250 lbs. per square inch.

The connections between the several elementary parts are

[1] This type of construction is covered by my U. S. Patents of March 4, 1902, No. 694,577, and April 20, 1909, No. 918,699.—Ernest L. Ransome.

easily taken care of; however, it is a common error to have the re-entrant angles square instead of chamfered, and this is of particular importance where the slabs rest on the beams or girders. The removal of the forms is greatly facilitated by having chamfered or beveled corners, and the finished structure is less likely to crack.

A special problem arises in connection with the exterior construction. In modern practice, the columns and floors are usually erected first, and separate curtain walls are next placed between the columns. These curtain walls may very well be utilized as deep beams at the same time by uniting the lintel below to the curtain wall in a substantial manner.[1]

The expansion and contraction of these walls is readily taken care of by setting their ends into recesses in the columns and the windows may be set into similar recesses above the curtain walls.

The cornice is tied to the roof structure by means of iron stubs projecting from the concrete.

A good design should show not only the location of all projecting stubs, ledges, recesses, etc., but also all the minor openings for heat and sewer pipes, and the location of pipes for gas and electricity. It is a surprising fact that those details are so much neglected; it is certainly much cheaper and better in every way to set proper sleeves, etc., for all such openings. Sometimes, the pipe-risers come up through the columns, but this practice is hardly to be recommended, as it is both unsanitary and makes repairs and alterations difficult or impossible. Special pipe shafts may be arranged for; or in some cases, the exterior columns are made large enough to accommodate the piping in cored flues of ample size. Heat flues or ventilation may be arranged for by having the exterior columns hollow with register openings leading to the several stories.

While the floors are usually covered with cement finish, wooden floors are in favor in many places. The floor is nailed to sleepers laid on top of the rough concrete base, and cinder concrete or stone concrete poor in cement is run between the sleepers. In many cases the sleepers apparently give good satisfaction, in others they are soon destroyed by dry rot. The

[1] This construction forms the subject-matter of my U. S. Patent, No. 694,580, March 4, 1902. — Ernest L. Ransome.

keeping qualities of wood embedded in concrete are not well known, but where the concrete is in direct contact with the wood, it must certainly act as a preservative if it has any action at all.

In hotels and similar establishments, linoleum or carpeting over a fairly smooth cement base should be very satisfactory.

As a general principle we must maintain that pipes should not be put inside the structural concrete, as they are practically inaccessible; the electric conduits may perhaps form an exception to this rule. After the steel has been placed the electrician places his outlet boxes and connects them up, so that the conduits rest immediately upon the steel. The slab must then be so thick that the entire pipe is buried below the neutral axis of the slab, as otherwise the strength of the slab is jeopardized. Note, however, that when the lights are suspended from the bottom of a beam, the outlet box and perhaps a short riser must be placed before the main tension steel is put in. Sewer or gas pipes should always be left exposed; sleeves are placed where they go through the floor, so that no cutting is required. Attention to all such detail goes a long way toward success in reinforced concrete work; if the plumber is turned loose in a building to cut whatever holes he may see fit he is almost certain to go through one of the main girders, steel and all. In fact, such a case came under the author's observation once.[1]

Where a wood-floor finish is placed over the concrete, many of the pipes may of course be concealed in the space occupied by the sleepers. Only risers and outlet boxes are then placed before the concrete is run. The specifications should state in detail who will furnish and set the various sleeves required, as there will otherwise be considerable friction between the several contractors.

A number of devices are on the market by means of which shafting may be attached at any place in the building. All such devices must be decided upon in advance and placed before the concrete is run. If a plain factory ceiling is all that is wanted, it is convenient to place suitable bolts at intervals, with their threaded ends projecting from the concrete; timbers

[1] In the Academy of Science Bldg., San Francisco, I once caught a plumber in the act of cutting off the brick corbeling on which the floor rested. Many such cases have come to my attention from time to time. — E. L. Ransome.

are then bolted to the ceiling wherever wanted, and the shaft-hangers attached to the timbers. All threads must be protected against concrete and rust. Sometimes the operation is reversed and tapped sleeves provided in the concrete, into which the necessary bolts are screwed. The head of the bolt projecting into the concrete should be enlarged so that the bolt will not tear out; the pressure may be distributed over a larger area by means of bars or plates underneath the head of the bolt.

Monolithic Construction. The monolithic building is usually erected a story at a time. First: the forms are set up, forming a complete wooden shell for the concrete to be deposited; next, the steel is put in place, and the concrete run around the steel and within the forms. Simple as this series of operations may seem, there are, nevertheless, a great many details to be attended to. This is particularly true with reference to the form work, which in itself absorbs a large proportion of the total cost of the building.

Forms. The simplest, but in the long run the most expensive method, is to cut the boards as needed and put them together box-fashion, nailing all the joints securely. Such forms cannot be removed without breaking the lumber to pieces and destroying a great deal of the concrete, particularly the corners, and at the present time no experienced worker in reinforced concrete would consider using such rough methods.

The first improvement consisted in making the slab-panel forms each in one piece, resting upon the form-panels for the beam-sides. All these panels had cleats nailed to the side facing away from the concrete, so that each panel remained a unit in itself throughout the erection of the building, and each panel could then be used over and over again. For long flat slabs, the panels rested upon joists, and sometimes it would even be necessary to shore the joists midway between the beams. For the shorter spans, up to five or six feet, the cleats used under the panels to hold them together would usually be sufficient. Figure 131 shows schematically the most essential parts of this arrangement, of which there is a very large number of variations. Usually, however, the parts are so arranged that the beam-bottom with the shores under same can be left in place while the panels are being removed, for the purpose of keeping the beams supported for a longer period than the slab.

Even when nicely adjusted, a falsework of this kind is soon destroyed by the continuous prying and pounding required to get it loose from the concrete, and the jarring and knocking

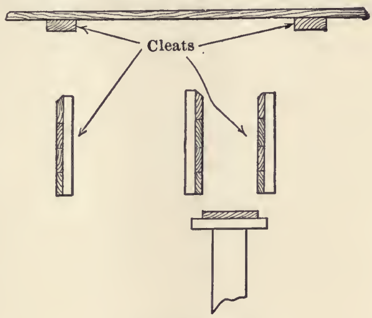


FIGURE 131.

about while shifting from floor to floor. An improved method of centering was therefore devised, whereby some of these objections would be overcome. This centering is shown in Figure 132, where the long box is split centrally down the middle, each half being held together by the triangular cleats, while the two halves are hinged together. The beam-bottoms rest

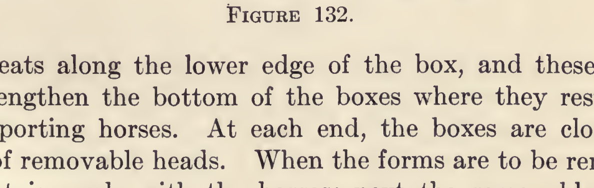


FIGURE 132.

upon cleats along the lower edge of the box, and these cleats also strengthen the bottom of the boxes where they rest upon the supporting horses. At each end, the boxes are closed by means of removable heads. When the forms are to be removed, the start is made with the horses; next the removable heads are taken off, and finally, the boxes are collapsed and removed. Of course, the sketch shows the essential outlines only; such portions as the stays for holding the boxes expanded, etc., have been omitted.

The main advantage of this arrangement rests in the fact that the benches or horses used for supporting the boxes form at once a safe and even foundation for the form work, and that the forms themselves are taken down, again put together, and erected by ordinary labor, there being no cutting or adjustment of any kind. Their use presupposes a standardized layout, and this, by the way, is a point to which altogether too scant attention has been given in the past. It is believed that, if a number of typical or standardized buildings are to be erected, the simple attention to duplication of parts may reduce the cost from 10 to 15 per cent., even if the buildings are at widely distant points. On the rougher and simpler forms of falsework it is frequently estimated that 60 per cent. of all lumber purchased is used on the job for which it was bought.

The great secret of success lies in attention to one fundamental point: that all parts must come easily apart when the forms are stripped from the concrete. Hence all shores must rest on wedges, and all joists, etc., must be keyed in place with wedges; wherever possible, bolts must engage in slotted holes from which they can be removed by simply loosening the nut without taking it off. Thus, for many purposes, the arrangement shown in Figure 133*a* is greatly superior to that shown in

FIGURE 133*a*.

FIGURE 133*b*.

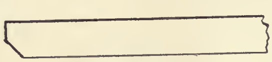


FIGURE 134.

Figure 133*b*, because a slight motion sidewise releases the bolt in A, while the bolt in B must be drawn through the hole.

The posts or shores should have one side of the bottom cut away at an angle, as shown in Figure 134, to facilitate removal.

Similar lines of argument lead to the result that all forms should be made with sufficient "slip" to leave the concrete readily, and that the re-entrant corners should be beveled; in short, what is good practice in making patterns for cast iron is also good practice in making molds for reinforced concrete.

Amongst the more common errors in the construction of forms, attention is called to the following:

Insufficient stiffness, so that forms sag or bulge.

Untight forms, so that cement is lost by leakage.

Irregular thickness or width of boards, so that bad-looking board-marks result.

Too tight fitting, necessitating crowbars and sledge-hammers when forms are removed.

Much time must be devoted in the office to the preparation of details of the forms and making up of lumber schedules; in fact, it would pay in many cases to have the forms made in a well-equipped carpenter shop and haul the forms to the job. Much time should also be devoted on the job to the inspection of the forms, both during erection, concreting, and removal, to insure against costly errors. But most of all, cleanliness must be enforced at all cost, so that no shavings or ends of boards find their way into the concrete.

In order to preserve the forms, the woodwork is frequently covered with crude oil, soap, or similar materials, and the results undoubtedly justify the expense. However, if the ceilings are to be plastered, no oil must be put on, as it prevents the adhesion of the plaster. In that case, the forms are simply given a good soaking with soapy water some little time before the concrete is run, and it must be admitted that the forms usually come away from the concrete as readily as when they are greased.

While the entire reinforced concrete floor in many cases may be stripped of all form work in a week's time after the concrete is poured, it is not always good practice to do so. In the summer, slab-panels and beam-sides may be removed in about one week, but the beams and girders should be shored up for at least three weeks. In the winter, the time must be extended considerably. Altogether, the removal of the forms calls for careful work when it is being done, and for discrimination as to the proper time. The strength of concrete depends greatly upon the nature of the materials entering into its makeup; hence what is safe practice in one place may be dangerous in another.

Reinforcement. We have considered the amount and requirements of the steel above; we shall here consider briefly the placing of the steel on the forms, and the preparatory work done on it.

Several devices of merit are on the market which facilitate the bending and shaping of the steel; the bending should be done cold and with so large radii in the curves that no injury results. While the difficulties incidental to bending heavy steel bars to sharp corners usually prevent such practice, it is different with the U-bars and other light steel, and many half broken bars of the lighter sections have without doubt found their way into important work.

Quite frequently the steel bars are assembled to suitable units, and from every point of view this practice must be recommended. It has, however, been argued that the assembling of the bars prevents each bar from sagging to its natural level, so that some bars are bound to be stressed higher and earlier than others. There is of course some truth in this, and perhaps some otherwise unaccountable cracks may be explained in this manner. The remedy is obvious — perfectly straight bars should be used only, but this follows from numerous other reasons as well.

The steel may also be bought ready-made, assembled in units. Owing to the cheapness of factory labor as compared with field labor, and to the better facilities found in a well-equipped factory, ready-made steel ought in many cases to be used with a considerable saving in money and time. However, the steel yard affords an outlet for the surplus labor, and for this reason it is often desirable to do the bending, etc., on the job.

In the early days of reinforced concrete, the beam steel was placed after a small amount of concrete had been run in the bottoms of the beams; similarly for the floor, the steel was placed during concreting. At the present time, the prevailing and better practice is to place all the steel in the beams and on the floor, and not to concrete before the steel has been inspected. Many errors and much poor workmanship are thus eliminated (Figure 135).

Means should be used for keeping the steel bars the proper distance away from the face of the form. Metal clips or cement blocks may be used, and are much to be preferred. The ordinary way is to have a laborer raise the rods from the forms with his shovel-blade or a hook made for the purpose, but such methods rarely result in satisfactory work. There are many and strong reasons for keeping the slab bars one inch from the face of the panel forms, and all other steel $1\frac{1}{2}''$ to $2''$ from the face of the concrete. The specified dimensions should be adhered to.

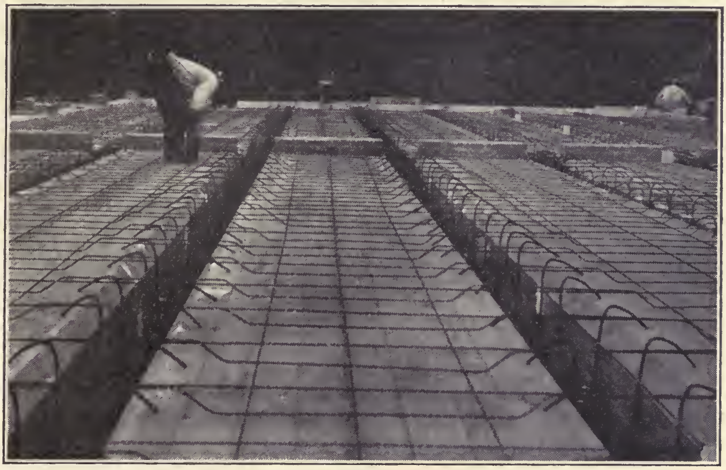


FIGURE 135. PLACING STEEL.
Morley Chemical Laboratory, Western Reserve University, Cleveland, O.
C. F. Schweinfurth, Architect; Alexis Saurbrey, Engineer.

Unit Construction. If we consider the list of patents given in a preceding chapter, we see that from the earliest days of the art, the method of casting the pieces in a yard and setting them when hard has been engaging the attention of inventors. Nor is this strange when we remember that our present "reinforced concrete construction" is a direct off-shoot of the artificial stone industry, and was originally introduced by men engaged in that kind of work not less than by men occupied in the manufacture of monolithic walls.

In the United States, the actual use of "Units" was not in much use before 1904, and the Textile Machine Works, erected in the winter of 1904–5 at Reading, Pa., was probably one of the first serious attempts. The Visintini System was used for the floors and girders, but the columns were apparently molded in place as in monolithic work; this building is

50′ × 200′, four stories high, and it is stated that the 2900 units were put in place for a total cost of $586.35 (labor only), which is only about 20 cents each. The completed building cost 7.7 cents per cubic foot.

In 1906 a one-story building was erected for the Edison Portland Cement Co. at New Village, N. J., the flat roof slabs were cast on the ground, on top of one another, separated by paper. After trying bare, oiled, waxed, and soaped paper, soaping just before casting was found best and most economical. The roof girders, each 50 feet long, were also cast in the yard.

The one-story building erected for the Central Pennsylvania Traction Co. at Harrisburg, Pa., in 1909, had roof girders about 37 feet long; these as well as the slabs were cast in the yard and set when hard. Another building of exactly the same dimensions and similar design had been erected close by several years before, by the monolithic method; it is stated that the saving in favor of the unit type was 15 per cent. in this case.

Recently the Unit Construction Co. of St. Louis has erected a number of buildings up to five stories high under the Unit System; the general design will be apparent from Figure 136.

In all the buildings just described, each member has been designed as an individual carrying element without assistance from the superimposed slab. This necessitates the use of T-beam sections in order to get the required compressive strength, or the use of extra deep beams or girders. In the system to be described below, the slabs are utilized in compression, and also used as an extra means of tying the entire building together, while in the cases just described, the pieces are tied together by virtue of bars projecting into pockets or open spaces in which concrete is poured.

In the Ransome Unit System, the beams, girders, and columns are made in the yard, but the slab is cast *in situ*. The first building so erected was the three-story office building of the Foster Armstrong Plant at East Rochester, N. Y., 1904–5, and the same method was subsequently used extensively in the United Shoe Machinery Co.'s plant at Beverly, Mass., for a group of four-story buildings, 60′ × 300′ in plan, and elsewhere. (See American Machinist, Sept. 7, 1911; E. L. Ransome: An Innovation in Concrete Building, from which the following is taken in part.)

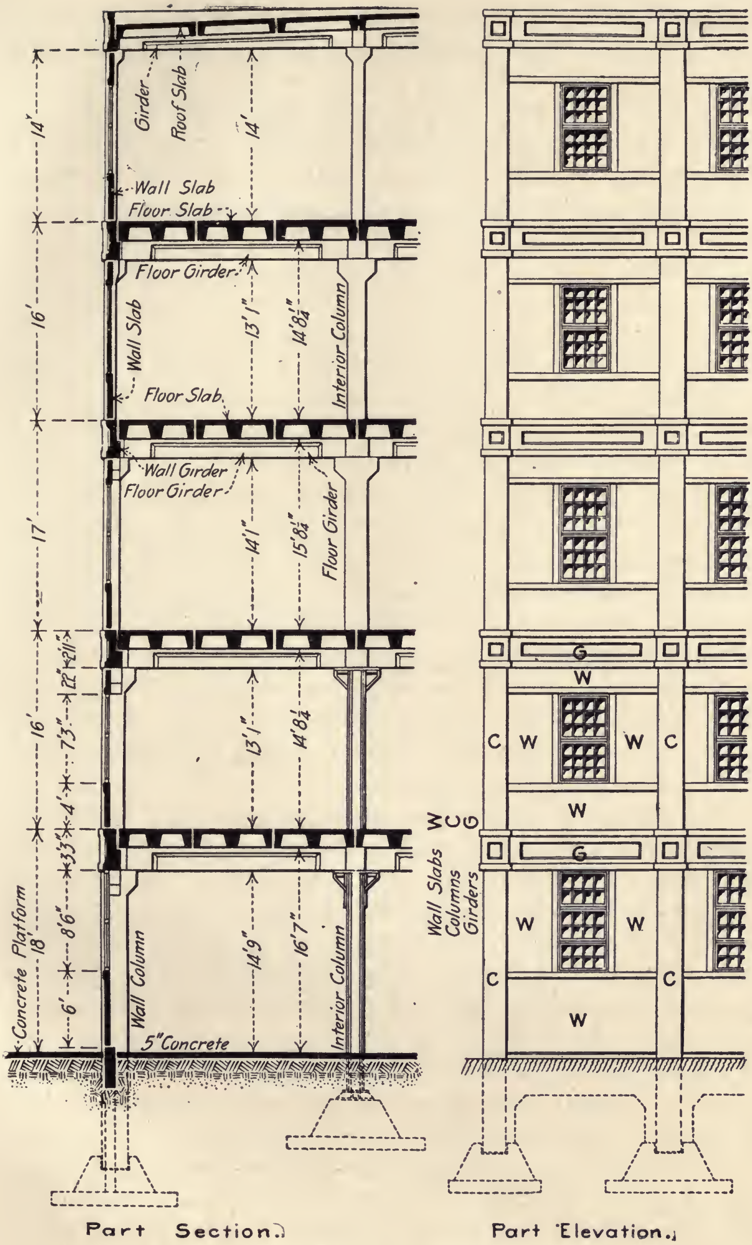


FIGURE 136.

Unit constructed building with separate slab-section reinforced with marginal and central beams. (From the Engineering News.)

Generally speaking, the beams and girders are cast of a depth equal to the distance from the bottom to the neutral axis only, and are provided with projecting iron ties. The slab forms are erected between the beams, which are usually spaced about 4 feet on centers, and rest upon 3×6 inch stringers bolted to the sides of the beams (Figure 137). The beveled corners of

FIGURE 137.

the slab mold bring the concrete down to the tops of the beams or girders. In the design, the U-bars must be made with a view toward creating the required tie between slab and beam. This is easily and economically taken care of. In addition, the beams and girders must be calculated to permit a working load equal to the weight of the forms, the wet slabs, the impact from concreting apparatus, and the like. When this is properly attended to there is no necessity for shoring of any kind; in fact, none is used. It is evident that the slab panels may be removed in a much shorter time when so constructed than would be allowable with the monolithic construction, because the old and properly seasoned beams take care of the entire load, up to the time when the full "live" load is brought on the floors.

The beams are mutually connected by means of tie bars placed in grooves in the tops of the several beams, and have vertical holes so that the hooked ends of the bars may engage in holes in the body of the beam. In the more recent developments of the system, the reinforcing rods project above the tops of the beams, and the union between the several pieces is effected simply by a loose rod inserted alongside the tops of the reinforcing rods and concreted in with them when the slab is run. In neither case is the beam considered as part of a continuous system, although with proper design the continuity might probably be taken advantage of to some extent.

The column rods are made discontinuous and the tops and bottoms of the column are enlarged so that the concrete alone will be sufficient to carry the weights at these points. That this method of construction is adequate, both for columns and beams, is amply demonstrated by the absence of vibration under heavy loads and high-speed machinery.

The column details are of considerable interest. Aside from the usual reinforcement near the sides of the columns, a longitudinal rod is inserted in a central cored hole extending lengthwise through the column. This hole runs from footing to roof slab. These rods, therefore, tie the columns of each story to those of the stories below and above. In order to unite the members, a thick, cream-like grout of 1:1 cement and sand is poured down this central hole, which is enlarged and flares out at the bottom, so that a secure and even bed is insured when the grout flows into this larger space.

One of the most important features is that of setting the majority of the pieces dry and grouting the joints afterward. That all joints really are filled is readily ascertained by the inspector who is instructed to see that a small surplus of mortar is forced out at the bottom of the joints. For this purpose small holes are left in the mortar with which the joints are calked. This mortar is a fairly dry and stiff mixture applied in the ordinary way with a trowel.

The buildings at Beverly are of rather complicated exterior design and a number of details, therefore, have been introduced, which would not be found in ordinary plain factory work. Thus, the very large flue columns in one of the courts are cast in place, as the weight of each exceeds the capacity of the derricks. Pockets and recesses are left in which the beams and girders are set. Otherwise, all the members are made in advance and set in place, with the exception of certain of the curtain walls which are cast in their final position and keyed to the columns with the ordinary recesses.

Figure 138 shows the principal details of a unit-constructed building erected under this system. Figure 139 shows some of the beams in the process of being set by the derrick. Figure 140 shows one of the stairs being set in place. One flight was built in place, another set of stairs was erected by the unit method with a saving of about 50 per cent.; the more complicated the required forms are, the greater is the saving.

The contractor's plant used at Beverly comprises an automatic mixing plant whence the concrete is discharged into an overhead hopper straddling an industrial track. This track parallels the building under erection and serves the purpose of bringing the concrete buckets of about one yard capacity to either the stiff-leg derrick used on the building proper, or to

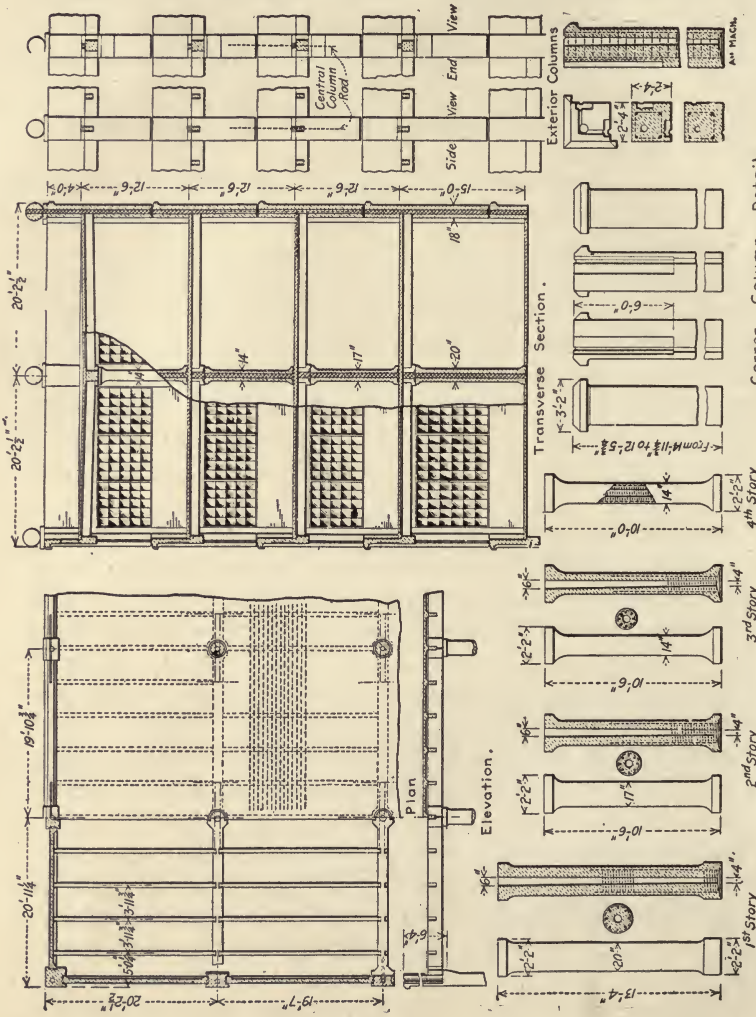


FIGURE 138. DETAILS OF THE "RANSOME UNIT SYSTEM."

the locomotive crane used in the casting yard. The columns are cast in gangs of four and other pieces in corresponding numbers as required. The side forms are removed in one to two days when the weather is warm, and the pieces are left undis-

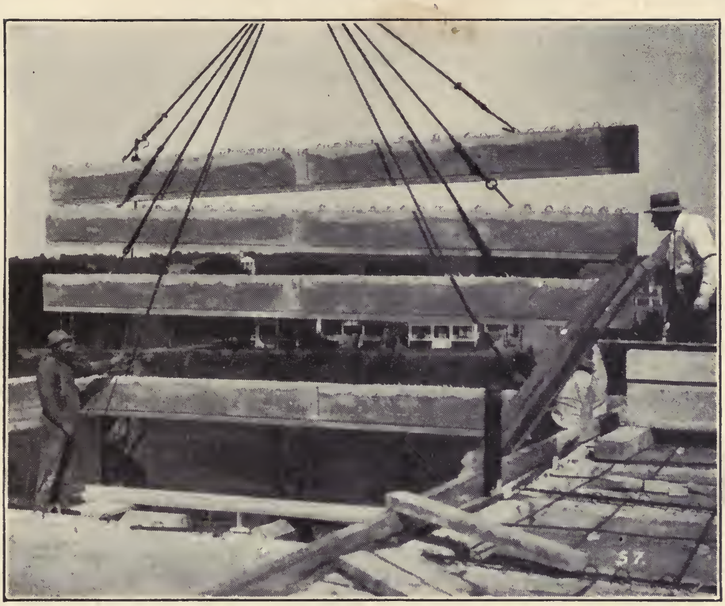


FIGURE 139. SETTING THE BEAMS.
United Shoe Machinery Co., Beverly, Mass. Ernest L. Ransome, Managing Engineer.

FIGURE 140. SETTING A FLIGHT OF STAIRS.
United Shoe Machinery Co., Beverly, Mass. Ernest L. Ransome, Managing Engineer.

turbed on their molding bed for about ten days. The periods are somewhat longer if the weather is cold.

If the pieces come without the reach of the stiff-leg derrick they are picked up and brought to the building by the locomotive crane. The latter serves a multitude of purposes in

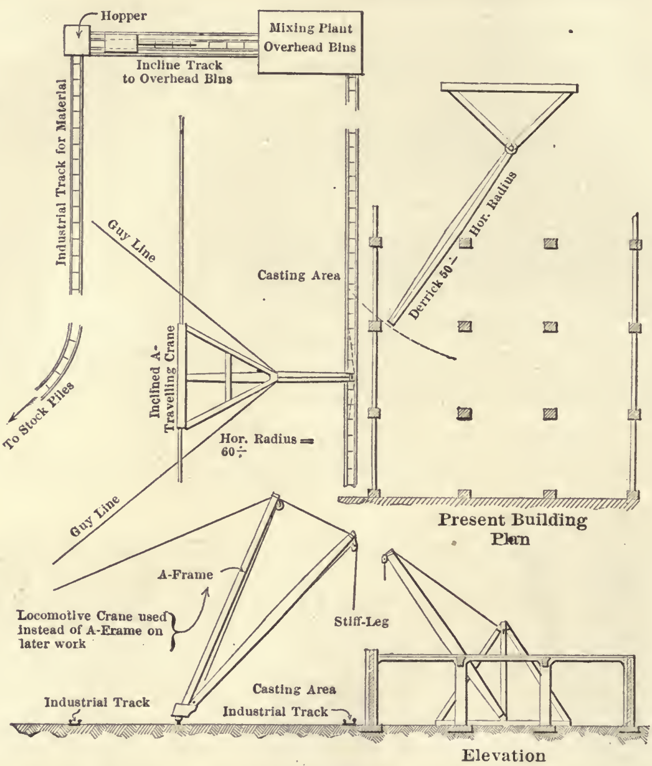


FIGURE 141. ARRANGEMENT OF PLANT, RANSOME UNIT SYSTEM. (FROM CEMENT AGE.)

stripping and moving the forms, concreting in the yard, and the like. One of the most interesting operations is the removal of the column cores. These are slightly tapering to facilitate drawing; they are six inches in diameter at the top, four inches at the bottom; they are made of wood and covered with sheet iron. In removing, the crane simply gives a slight pull on the core, which comes out easily if the concrete is fairly green. No instances have been recorded where a column was in the least injured by this treatment.

The concrete is usually handled in one-yard bottom-dump buckets. The molding of the units presents very few difficulties, and the workmanship is greatly superior to that obtained with the old method. This high standard is evidenced in all the work; the lines are straighter and the work generally truer, than can possibly be obtained commercially by the older methods.

FIGURE 142. SETTING A SLAB.
United Shoe Machinery Co., Beverly, Mass. Ernest L. Ransome, Managing Engineer.

It is practically impossible to produce a large monolithic reinforced-concrete building commercially without some indications of bulging forms, or of supporting shores having been carelessly wedged up, or corners fractured in prying the forms loose. There are no such troubles on work of this kind, because there are no shores to give way, no forms to bulge.

It is quite common to erect a space of floor (for a height of one story) 60 feet wide by 40 feet long in one working day, including the setting of slab forms and pouring the concrete. This also allows time for calking the joints and pouring the grouting into the cored holes and recesses.

In one of the one-story buildings erected at Beverly, the exterior walls were made of 3″ concrete panels, reinforced with $\frac{1}{4}$″ twisted bars vertically and horizontally spaced about two feet apart. These panels were set when eight days old, some of them with door or window openings, or even with the windows concreted in place (Figure 142). The wall panels were cast on top of one another in stacks, and the layers separated by means of a heavy coat of common lime-whitewash.

In all unit work, a proper margin must be allowed for in the design; all horizontal pieces are made from $\frac{1}{2}$″ to $\frac{3}{4}$″ short and all pockets or recesses are extra large; the openings are filled with grout afterwards. Two transits were generally used when setting and plumbing the columns.

CHAPTER XII

FOUNDATIONS AND PILING

Foundations. The reinforced concrete footing in common use is simply of pyramidic shape with the top removed. The discussion given in Part II will suffice for the design of ordinary footings; where two or more adjacent footings are merged, the design may be of either the flat slab type or it may embody the slab and beam principle.

Quite commonly, the hole or trench is excavated to the approximate size of the footing, and the concrete dumped in the hole without forms of any kind. Such practice is not to be recommended unless the ground is very stiff. On the contrary, as the stability of the entire building depends upon the integrity of the foundation, the greatest care should be taken both with the forms for the outside, with the banks of the excavation so that dirt will not fall into the footing, and with the proper placing of the steel. The latter should be protected with not less than 4″ of concrete, preferably more, and the concrete around the rods should be rich in cement and dense, so as to exclude water.

If metal base plates are used under the columns, special care must be taken to prevent hollow places under the plates. This is best accomplished by setting the plates in a thin grout 1: 2, examining afterwards each plate by pounding with a hammer.

Reinforced concrete footings cannot conveniently be put in under water, so in a wet excavation it is advisable to use plain concrete footings of ample dimensions to meet all emergencies.

Piling. A number of patents exist covering the various methods of manufacturing concrete piles, and several of these are operated by companies making a specialty of concrete piling. To name a few examples:

The Chenoweth pile, made by rolling a sheet of fresh concrete, with fire-fabric reinforcement, around a central reinforcement or tube.

The Raymond pile, cast in a thin shell of steel, which latter is driven by means of a collapsible pile-core. The shell remains in the ground.

The Simplex pile, cast in a cylindrical shell strong enough to stand driving and withdrawing, leaving, however, the point or shoe behind.

It seems, however, that two methods are open to the public: (1) To drive a pile, withdrawing it, and then fill the hole with concrete; and (2) the use of concrete piles molded in advance and driven as wooden piles. The first of these methods is open to several objections, so we shall here give an account of a pile-driving job according to the second method.[1]

The piles support a one-story building with 45-foot roof spans resting upon exterior piers. Underneath each pier three piles were used; in addition, the chimneys and other foundations rest on piles. A total number of 484 piles 10″ × 10″ and 13 feet long were required, driven through fill and bog into tenacious blue clay. The penetration into the clay was from 2 to 3 feet. The piles were cast in the yard (Figure 143); the ground was levelled and tamped, then covered with a layer of sand, one inch thick, and re-tamped. On this bed the piles were molded, the sand forming one side of the mold. Two sides were formed by surfaced boards; the 45° pointed end was made by simply filling in with molding sand to the required slope, by two $1\frac{1}{8}''$ thick pieces for the sides, and by trowelling the top down to the required angle. Otherwise, the surface of the pile was smoothed with the back of a shovel. The forms were removed in sixteen hours and immediately set again; thirty piles were cast in one operation in a gang mold.

The concrete was mixed in the proportion 1 : 1 : 2, using clean bank sand and crushed trap rock, pea size. This concrete had an average compressive strength of 97 tons per square foot at seven days when tested in the compression machine. It was found, however, that a 1 : 2 : 4 mixture gave an average strength of 104 tons per square foot in seven days, using 2″ rock. This concrete could have been used successfully and would have saved $1.03 per pile, reducing the cost from $6.63 to $5.60.

[1] From a report submitted by Mr. B. C. Gerwick, who acted as resident engineer on the job referred to.

The reinforcement consisted of 4 — $\frac{1}{2}''$ square twisted rods, and a spiral reinforcement of $\frac{1}{8}'' \times \frac{1}{2}''$ hoop iron, 4″ pitch. Experiments were made with No. 6 and No. 8 wire of same pitch, but such piles did not seem to stand the driving as well. In either case, a $\frac{1}{2}''$ twisted steel bar was used as an extra collar reinforcement near the head, and the point also had an extra reinforcing bar of same section.

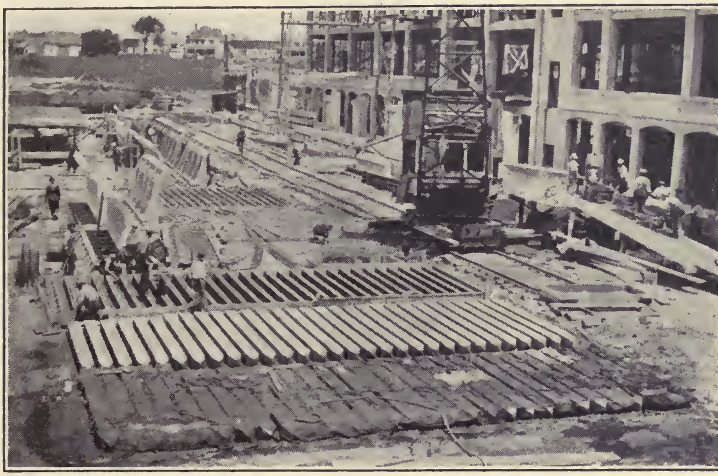


FIGURE 143. MANUFACTURING REINFORCED CONCRETE PILES.

United Shoe Machinery Co., Beverly, Mass. Ernest L. Ransome, Managing Engineer.

The sand in the bottom of the form was first tamped and sprinkled; two inches of concrete were carefully placed, the reinforcing cage put in, and the balance of the concrete placed at once. As the pitch of the hoops was 4″, they offered little obstruction to the concreting. When the piles were lifted the bottoms proved to be smooth and the sand did not adhere to the concrete.

The fresh concrete was covered with old sacks and kept damp for three or four days. In loading, the pointed end was raised with a bar, a rope sling slipped underneath, and the pile put on the stone wagon by the locomotive crane available in connection with other work. Four piles, each weighing about 1350 pounds, constitute a load, and the haul is about one-half mile.

A drop-hammer weighing 1600 lbs. was used in driving. On top of the pile a cushion of several layers of old fire-hose rubber and felt was placed, and over this again a 5″ cast-iron block. An oak follower four feet long rests on this block, and takes the blow of the hammer. The follower must be of good quality with both ends banded. The fall of the hammer under the last blow was from 10 to 20 feet, with a penetration under the last blow of about $\frac{3}{4}$″. On an average, it required about seventy blows to drive a pile which would penetrate from three to four feet into the clay. One pile was pulled by means of a lever and found to be in perfect condition. The piles are driven when eight days old.

The crew consisted of foreman, engineer, four pile driver-men and two laborers. This crew, including the use of the pile driver, was hired for $30.00 per day at eight hours. The piles were, as stated above, usually in groups of three, the distance between two adjacent groups being 16 feet, so that when three piles were driven, the driver had to be shifted 16 feet. The average time of the shift was 23 minutes. The repairs, etc., totaled 15 minutes per day, and the average time consumed in actual driving was 12 minutes. The total average time per pile was about 20 minutes, or 24 piles put in every eight hours.

COST OF PILING

484 piles, each 13′–0″ long 10″ x 10″ in section

			per pile
Grading casting yard for bottoms	$25.45		$ .053
Cost of gang mold for 30 piles			
900′ B.M. spruce @ $21.00	18.90		
Nails, 10d	.20		
Labor, carpenters @ 47$\frac{3}{4}$c per hour	26.65	$45.75	.095
Setting forms, per gang of 30	7.00		.233
Stripping forms, per gang of 30	.72		.024
Cleaning and greasing, per gang of 30	1.16		.038
Placing concrete, per cu. ft.	.0235		.20
Mixing concrete, per cu. ft.	.022		.187
Labor on reinforcement			.66
Cement, stone, sand, and steel, cost			3.517
Hauling $\frac{1}{2}$ mile: Loading 4 with crane	.26		
Hauling 4	.37		
Unloading 4	.11	$ .74	.185

(Cost of Piling — Continued)

Driving: Crew under contract at $30 per day,	
Average day's work, 24 piles	$1.25
Coal, oil, and grease	.125
Cushion cap, $31.40	.065
Total cost per pile, in place	$6.632
(Overhead charges not included.)	

Where a large number of piles can be made in a centrally located yard, it sometimes pays to cast the piles vertically. Round forms can then be used as well as square.

A permanent plant of this kind exists at Cleveland, Ohio, as perhaps elsewhere.

CHAPTER XIII

FINISHING OPERATIONS

Corners. Square corners are contrary to the nature of concrete. Projecting corners are difficult to make in the first place, as the concrete seldom penetrates to the very apex of the angle; in the second place they are liable to injury both when the forms are removed and while the concrete is green. Once broken, they cannot be repaired so that the patch looks like the balance of the work. The re-entrant corner is easily made, but objectionable for the reason that a sharp dent in the concrete very often forms the starting-point for a crack which might otherwise have been avoided. This is explained by the same observations made in regard to cast-iron; in addition the form is often locked to the concrete by a sharp corner so that the workmen use too much force in removing the forms. Broken corners are the great drawbacks in concrete construction; they may easily be avoided by chamfering the forms so that all sharp angles are excluded.

Flat Surfaces. All the defects in the form work will show on a flat concrete surface; in addition, all defects in the concrete will show. It is difficult if not impossible to make perfect forms; it is practically impossible to maintain the forms in perfect condition, because the water in the concrete is absorbed by the wood in the forms, causing swelling and warping. The marks left by the forms are called "board marks"; if a finished piece of work is desired the board marks must be either concealed or erased. In the first place, the concrete work is faced with various materials, such as brick, terra cotta, plaster, etc.; in the second case, the surface itself is improved by tooling, rubbing, or brushing.

Plastering. Plaster usually comes off again sooner or later, especially on outside work. It should be used for indoor work only, and then only in emergencies; if plaster is insisted upon, the ceilings should be burned with acid, and the form work should be made as rough as possible. In that case, very fair results may be obtained, but plastering always remains an art more than a science, so that skilled labor is a most essential feature. There are various plasters on the market, made especially for concrete surfaces. A great deal of satisfactory work has been done with such material, but its general use is still too recent to warrant absolute confidence.

Tile-concrete construction is well adapted for plastering.

Brick and Terra Cotta Facing. The concrete must be true to line and level, as it is otherwise difficult to put on the brick facing, and impossible to put on the terra cotta facing. For brick, galvanized wire wall ties are left projecting from the concrete, say 12″ apart diagonally, and bolts are placed in the concrete to receive the angle irons which carry the brick work over door and window openings. For terra cotta the arrangement of the ties and supports varies greatly with the design. Usually a hollow space is left between the concrete and the terra cotta facing; cement mortar deposited in this space ties the facing to the concrete behind, as the facing blocks have projecting ribs on the back, while iron anchors project from the concrete, so that the whole is locked securely together. At intervals supporting ledges must be arranged to transmit the weight of the facing (which is considerable) to the structural concrete. It is advisable to make all ties of soft iron so that they will not break when adjusted. It is very important that suitable play be provided for, as neither brick nor terra cotta can be made to exact dimensions, while the concrete construction is very apt to vary slightly from the specified dimensions.

Improved Surfaces. The removal of the board marks is possible under one condition only, and that is, that all joints between boards must be tight. The joints fill with the finer parts of the mixture, especially with cement, so that the small ridges between the boards are rich in cement. It follows that the concrete immediately behind the ridge is leaner in cement than other parts of the surface, and it is therefore softer than the surface generally, so that any mechanical treatment of the surface removes too much at the ridges, forming small grooves looking almost as bad as the original ridge. Hence the joints must be tight, so that no cement can ooze out, and fairly smooth, so that few and small ridges only are formed; the carpenter work must be good, the forms must be made of good lumber and nailed securely to the cleats to prevent spring. The completed form must be coated with grease, vaseline, crude oil, or, best of all, a cheap grade of black japan. This kind of work is expensive when undertaken on a large scale; to secure the forms which retain the concrete so that there will be absolutely no deflection, is not easy. When now the forms have been completed to the satisfaction of the engineer the concrete is deposited at once, as sun and wind will destroy the best-made piece of form work. The concrete must be placed with the greatest care, as every defect will show in the finished work. The concrete mixture must be so gauged that there is a surplus of mortar; usually 1: 2: 3½ or 1: 2: 3 will be found suitable. A mortar facing run in the form with the body of the concrete may be used on work of very large dimensions only, as it is otherwise practically impossible to deposit the mortar. But a mortar without stone looks very dull when tooled.

Tooling. The surface is bush-hammered either by hand, or, preferably, with a pneumatic tool. An ordinary chisel may be used, but special tools are sold for this purpose. The defects

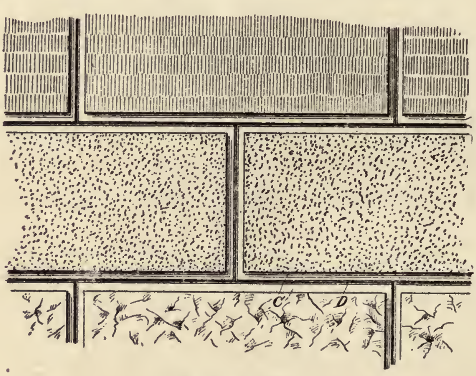


FIGURE 144*a*.

in the concrete are brought out strongly by this method, and repair work looks very bad, especially in rainy weather. But if the concrete was put in right in the first place the effect is very pleasing, and for large surfaces tooling must be considered as the most satisfactory and most pleasing finish (Figures 144*a* and *b*).

Rubbing with carborundum blocks (or similar hard material) is very expensive. The form work must be absolutely first-class, and the concrete must be very hard before rubbing is attempted, but the results justify the expense. The surface is removed to a depth of $\frac{1}{4}''$ to $\frac{3}{8}''$; the grain of the concrete is

FIGURE 144*b*. GIRLS' DORMITORY, LELAND STANFORD JR. UNIVERSITY, PALO ALTO, CAL.

The exterior construction and the floors are of reinforced concrete. Ernest L. Ransome, Engineer.

thereby exposed with a smooth and glossy surface. It has been found most satisfactory to rub the concrete down dry, in the cases of ceilings and similar surfaces where large quantities of water cannot be applied; certain kinds of cement floors are manufactured in the same way, but by a wet rubbing. When the concrete is to be painted, a very good surface may be had by this method by taking only the board marks off, leaving a practically, but not entirely, smooth surface (Figure 145).

Brushing. The forms are removed as soon as feasible and the green surface brushed hard with wire brushes so that the mortar between the stone aggregate is removed. Plenty of water is used in this process, and the stones finally show in relief on the dull gray mortar backing. A sparkling, many-colored aggregate is used, and the effect is very good, although perhaps a little artificial.

Unfinished surfaces are sometimes used for factory buildings, stables, etc., and all degrees of work, from very good to very poor, may be found. Occasionally an effort is made to improve such surface by rubbing cement mortar into the pores at once upon removal of the forms, and then rubbing the entire surface down with cement bricks, or sometimes hardwood blocks. If the purpose is to fill the pores only there can be no criticism of this method, provided all the surplus mortar is again removed;

FIGURE 145. MONOLITHIC CONCRETE STAIRS AND RAIL, CAST IN ONE PIECE THREE STORIES HIGH. CONCRETE WAS RUBBED WITH CARBORUNDUM AND PAINTED.

Morley Chemical Laboratory, Western Reserve University. C. F. Schweinfurth, Architect; Alexis Saurbrey, Engineer.

often the mortar is allowed to remain on the surface as a thin film, in which case more or less pealing is bound to follow. Many arches and abutments throughout the country have been provided with a coat of this kind, and there is hardly any locality where samples of this work may not be found, showing the disgraceful results obtained.

Cement Finish. Only in exceptional cases may the rough concrete floor be used, partly because the surface is too coarse, partly because it wears out too rapidly, partly because it forms part of the structure itself and therefore needs protection against wear. The rough floor is therefore covered with a sheet of cement mortar, called "cement finish." It may be applied to the green concrete surface as soon as it is hard enough to allow walking on it, or at any time afterwards. In the first case a good bond is assured by simple precautions, such as removal of the "slam," a white scum forming on top of concrete laid with an excess of water. If the concrete is hard and old, the surface must be cleaned with muriatic acid, water, and scrubbing brushes, as no cement finish will stick to a dirty surface. The finish is put on in a rather dry condition, like soft dough; about $\frac{3}{4}''$ or $1''$ thick as a minimum, and up to $2''$ thick as a maximum. It is trowelled to a hard surface in order to ensure good wearing qualities, and it may be divided into panels or not, according to circumstances. The object in dividing the surface into panels is purely ornamental; cracks may be avoided by dividing the base as well as the surface into suitable blocks. A structural reinforced concrete floor is not readily divided in this manner; in fact, one of the objects of good design is to make the floor continuous as far as possible. It is therefore proper to divide basement floors, sidewalks, and similar pieces into blocks by deep and wide separations, while the finish on a reinforced floor may as well be laid in one continuous sheet. Thereby is also avoided the breaking of the edges of the individual blocks, so likely to take place under heavy trucks in warehouses. A surface laid in this manner will show all the cracks in the base below, and for this reason as well as on general principles, all care should be taken to avoid cracking. These are, as stated above: proper arrangement of principal and secondary reinforcement, bevelling of all re-entrant corners between beams and slabs, protection against wind and sun, liberal sprinkling, and avoidance of premature loading and jarring. Great care should be taken when the forms are removed; in fact, a large number of "unaccountable" cracks are due to carelessness in removing the forms.

To avoid cracks entirely is hardly possible, especially in tile-concrete floors, where top cracks are very frequent, due to the unyielding nature of the tiles. Such floors are better provided with a wood floor on top of the concrete.

The mixture used for the finish should be one part of cement to two parts of selected sand, whereby is meant a sand with particles well graded from fine to coarse, clean, and sharp. The largest particles should not exceed the 1/4″ to 3/8″ ring. The finer the sand, the easier it works under the trowel, so that fine sand is the preference of the cement finisher, to the injury of the work. Aside from proper materials, skilled cement finishers are indispensable to good results. The engineers' supervision of the workmanship is usually confined to the results, as few engineers are sufficiently well posted on cement finish to supervise the details of the workmanship.

CHAPTER XIV

FIREPROOFING AND FIRES

No building is absolutely "fireproof," and the most that can be accomplished is to retard the spread of the fire to such an extent that the fire can be brought under control before the barriers are destroyed. But this is only one side of the question, for in many cases more damage is caused by smoke or water than by the fire itself. To prevent the smoke from penetrating to portions of the building not affected directly by the fire, is usually impossible, but much may be done to prevent the water from leaking down into the stories below the fire. We encounter here a much neglected problem: In most cases, pipes for heat, sewerage, etc., are carried through the floors by means of open sleeves, and the water naturally finds its way out through all these holes in the floors. However, if we consider a perfectly waterproof floor without means of escape for the water, the load on the flooded floor might easily exceed the capacity of the structure to a dangerous degree.

While therefore many owners of reinforced concrete buildings carry no insurance on the building itself, it is not advisable to neglect the insurance on the contents, except where they are of such a nature that they are not easily injured by water, smoke, or heat. Much will also depend upon the character of windows, partitions, stair- and elevator-wells. In the majority of cases, the so-called "fireproof" building is equipped with wood trimmings, plain glass in wooden casings, and has a wood floor over the concrete base. While in such cases the reinforced concrete escapes injury, the contents are usually a total loss, frequently with loss of human lives. There is, without doubt, room for great improvement along these lines. The arrangement of these matters, as well as those pertaining to stairwells and elevator openings is, however, beyond the scope of this book.

Turning now to the concrete itself, it is admitted that no material is absolutely fireproof, and concrete as well as other materials must finally fail under a long and severe fire test. But each particle in the crystallized concrete contains chemically bound water which is given off under high temperature, and the temperature of the concrete itself is thereby prevented from reaching a high intensity throughout the mass. Concrete itself is a good conductor of heat compared with the true insulating materials.

The concrete surrounding the steel must be made so thick that a large part of it may lose its water (and thereby its strength) without injuring the strength of the concrete touching the steel, as otherwise failure would result. But large amounts of concrete are expensive: practice has therefore settled upon 2″ of protection on columns and girders or beams, and 1″ on slabs. These thicknesses are entirely arbitrary and may be varied according to location and exposure, but it will be seen that they are also structural minima, and that with less concrete around the steel there can be absolutely no bond, and a small enough factor of safety as far as the workmanship is concerned. It happens quite often that these specified minima are still further decreased by carelessness on the part of the concreting gang, so that rust spots show through the concrete, or the steel is even exposed to view. Such conditions are always indications of workmanship of the poorest class.

The parts most exposed to the attack of fire are the projecting corners. Experience has shown that rounded or chamfered corners are much less liable to attack than a plain square corner; in addition, square corners are almost always more or less fractured when the forms are removed. In warehouses or other buildings where heavy stuff is handled, the lower part of the columns should have extra protection, such as angle iron guards for the corners, or even an iron mantel surrounding the concrete entirely. The same is true of thresholds and stairways; the steps of the latter are often protected with some patented metal covering, of which the nosing piece is the most essential part. The elevator hatches should also have a proper protection; angle iron guards are easily fastened and effective.

In addition to these obvious safeguards, we have an excellent method of increasing the fire resistance of concrete. It simply consists in adding a small amount of salt to the water with which the concrete is mixed. This fact was proved in a peculiar manner: During the erection of the Bayonne, New Jersey, warehouse for the Pacific Coast Borax Co. in the winter 1897–1898, experiments were made with salt as a frost preventive, the work being carried on in very severe weather, with temperatures sometimes below zero. In 1902, the building went through an exceptionally hot fire, started from a burst oil main in the basement, which soon was flooded with burning oil. On the upper floors, combustible materials of all kinds, including heavy barrels and boxes, added to the fire, yet the concrete came out of the fire with hardly any damage, and the concrete work of the entire building, 200′ × 240′, and partly four stories high, was repaired for less than $1000. Quantities of fused cast-iron from the machinery and copper from the dynamos and motors were in evidence after the fire (See Iron Age, May 28, 1902), showing that the fire must have been unusually hot. (Figure 146 shows a block of fused cast-iron from this fire).

The increased fire-resistance due to an admixture of salt has also been demonstrated on test specimens made for the purpose.

The general behavior of reinforced concrete in conflagrations has been very satisfactory in the Pittsburgh and Baltimore fires, where but few reinforced concrete buildings were within the fire-swept area; the most convincing proofs were however furnished in the San Francisco earthquake and conflagration, where buildings of all kinds suffered, but those of reinforced concrete less than any others. Tests without number have been made to determine the conductivity and fire-resistance of concrete. As a result, it may be stated that the better the concrete is made originally, the better it will be adapted for fireproofing purposes, and a four-inch concrete wall may be exposed to the hottest fire for hours, on one side, while the other side remains so cool that the hand may be placed against it without fear. The use of reinforced concrete for heat flues and chimneys is justified from this fact.

At the present time, a " fireproof " floor may be built in one of three ways: (1) By combining steel and concrete, whether the steel be in the form of a reinforcement, or as an independent skeleton. (2) By combining steel and tile, in which case the steel forms the well-known skeleton so commonly used in the modern skyscraper. (3) By combining steel with both tile and concrete in various ways. (There is indeed a fourth method, by suspending brick arches between steel beams, but this is in little use at present except for special structures.)

FIGURE 146. FUSED CAST-IRON PULLEY FROM THE BAYONNE FIRE.

The competition is therefore between concrete and hollow tile, both or either in combination with steel. Wherever put to the test, concrete appears to have carried the day. Two reasons suggest themselves for this fact: (1) The expansion of concrete and of steel is practically the same, while the expansion of tile is different from that of steel, so that there is a tendency to readjustment under fire in the latter case, and none in the former; and (2) the hollow tiles in common use are very poor conductors of heat, so that, while the steel is protected in an excellent manner while the tiles stand up, the unequal expansion of the several parts of the same tile causes the lower flange to break off, especially when suddenly cooled, thus exposing the steel. The proofs of this statement are ample and convincing and may be seen by reference to the photographs in the Governmental report on the San Francisco fire.

It must be understood that lath and plaster construction is not included as a fireproof material, and has no value as such.

In closing this paragraph, we must call attention to the ever present danger attending the use of reinforced concrete buildings veneered with brick or similar material, where the horizontal supports are exposed over the window openings as is nearly always the case. A hot flame through the window would probably injure the supports and wall-ties sufficiently to cause parts of the veneer to fall, although no such accidents have ever come to our attention.

Other considerations also lead us to doubt the continued stability of thin veneer walls, and there seems to be no good reasons for their extensive use.

CHAPTER XV

REPAIRS TO EXISTING BUILDINGS

The general wear and tear on a well-constructed reinforced concrete building is insignificant and confined to the finish coat of the floor. The repairs consist in careful removal of the worn surface, thorough cleaning of the floor, eventually with weak muriatic acid, — the application of a bonding substance such as Livingstone, Ransomite, or similar, and the placing of a new surface coat.

It happens, however, occasionally that carelessness when the work was made causes trouble, and a brief description of some cases of this kind may be of interest.

Cracking of the floor slab may be due to a number of causes: concrete poorly proportioned with accompanying excessive contraction, too rapid drying out of the concrete, etc. All cracks may be repaired that are caused by a natural adjustment when a stable condition has been reached, by simply cutting out the cracks, dove-tailing the bonding surface on both sides, and filling in with fresh concrete. Many slabs are broken when the forms are removed, although the crack does not appear for some time. Cracking of beams or girders is usually due to careless or premature removal of the forms. A gaping crack is an indisputable sign that the reinforcement has slipped, and it is then a question of removing the entire beam and putting in a new one. In that case, pockets are left for the new beam at each end, and the new concrete tied as well as possible to the old work.

If upon examination the steel is found too high in the beam, as sometimes happens, it is possible to cut away the bottom portion of the beam for its entire length, and to put in a new bottom with proper reinforcement, leaving the old steel in place. The new bottom is tied to the old beam by means of frequent U-bars which are concealed in vertical grooves cut for the purpose in the sides of the beam; the upper ends of the U-bars are carefully anchored in new portions of the slab inserted in spaces made to receive them. The new reinforcement must extend well over the supports, and firm anchorage must be provided for it. Hollow spaces in the columns are repaired by cutting the poor concrete entirely away, cleaning the surfaces, and pouring new concrete in. It must here be observed that the concrete surfaces are made to slope to such an extent that all air can escape, preferably through a vent on the opposite side of the funnel through which the new concrete is poured. The purpose of the vent and funnel is to make certain that the concrete fills all cavities by putting the fresh concrete under pressure. The surplus stuff is dressed off and the surfaces smoothed down.

Under no circumstances should cutting in concrete be done except in the presence of a reliable engineer who understands the structural importance of each member, and proper shoring must be put under the beams, etc., before the cutting is proceeded with.

When cutting holes in structural concrete of any kind, the lighter hammers and chisels should be used in preference to the heavy tools, and many light blows rather than a few heavy ones should be insisted upon for the reason that heavy blows have considerable shattering effect on the concrete, especially in the first few weeks after pouring. Hence the pneumatic drill is to be preferred where obtainable, even if at much greater cost, especially when putting in new bolt holes, etc., in great number. But drilling into the bottoms of beams and girders, or into the sides of hooped columns, should not be allowed when avoidable, and it is quite often possible to confine the drilling to the slabs and the sides of the beams.

Sometimes, an annoying and troublesome condition arises from the fact that the laitance, or dead cement, accumulates in the beam-bottoms. This can happen only where the concrete has been made with a surplus of water, and the water has been allowed to run ahead of the concrete into the bottoms of the beams, carrying considerable amounts of cement with it. The water and cement form a soapy, white substance which never sets up, and, after a while, large cakes drop from the beam-bottoms, sometimes an inch thick. The only efficient manner of repairing is by putting in a new beam-bottom, tying the new concrete to the old, and this procedure is very expensive, although cheaper in the long run and far better than repairs with cement mortar troweled on.

In exceptional cases, poor foundations cause unequal settlement and cracks. If the footings finally adjust themselves to a permanent level, the cracks may be repaired as described above; but even in that case, the building has lost considerably in carrying capacity, especially if constructed with continuous beams and girders. In one case, it became necessary to install new footings, columns, and girders alongside the old work, but in that case, the original footings had not been brought down to the proper level, and the girders had been erroneously designed.

After-treatment of the surface of cement finish may be desirable in exceptional cases to further the hardening, in which case a wash of equal parts of water and the commercial solution of Silicate of Soda is applied. Silicate of Potash may be substituted for the Soda, but the best results are obtained by using a wash of two parts of water with one part of Silicate of Soda and one part of Silicate of Potash; the two latter in the ordinary commercial solution.

A final wash with Chloride of Calcium is very desirable, especially if there is no free lime in the cement.

For repairs to small cracks, a mixture of Silicate of Soda and Chloride of Calcium may be poured into the cracks, but as this solution sets very rapidly, Alum may be used instead of the Chloride of Calcium as this mixture sets much slower.

CHAPTER XVI

ACCIDENTS

A GOOD reinforced concrete building is as permanent as any type of construction known today, and where a building of this kind has been taken in use, it has never been known to fail, with one or two exceptions where the design was faulty, or where the foundations were entirely inadequate. In the very few cases where reinforced concrete buildings have been purposely demolished the task has proved an arduous one, as for instance the seven-story building of the Baltimore News which was taken down in the spring of 1911 to give room for a larger structure.

It would be possible to enumerate a number of minor mishaps, serious enough to those whom they affected, but similar to those which do occur in all lines of building construction, whether brick, steel, or concrete. Here we will limit ourselves to the few disasters which attracted universal attention, and give a brief account of the cause in each case, in so far as the cause is known. It will be appreciated that the tangled mass of débris, and the more or less colored account of the actual conditions given by the parties directly affected, furnish but poor material upon which to base an unbiased opinion.

The collapse of a portion of the Amsden Block at South Framingham, Massachusetts, in July, 1906, has been traced to the settling of the foundations, and inasmuch as the interior construction consisted of reinforced concrete only for the slabs and fireproofing, the beams being of steel, and the columns of cast-iron, there is no reason to believe that the reinforced concrete was to blame for the failure.

The Bixby Hotel, at Long Beach, California, was a building H-shaped in plan, with reinforced concrete construction of the tile-and-concrete variety for the interior, and the usual concrete skeleton for the exterior construction. A large portion of the bar of the "H" fell while the roof was being concreted, on November 9, 1906. Questions as to design and unit stresses assigned to the columns have been raised, and it seems probable that some of the columns failed in one of the upper stories. However this may be, premature removal of the falsework undoubtedly entered into the causes of the collapse.

The Eastman Kodak Company's building at Rochester, New York, was partly demolished by a sudden failure on November 21, 1906, while the waterproofing was being put on the roof, which at that time was seventeen to eighteen days old. The initial failure seems to have been traced without doubt to column No. 47 which at that time was about three weeks old, but there also seems to have been more or less neglect on the job with reference to the proper placing of the column reinforcement and some of the columns had considerable amounts of saw-dust and chips of wood embedded in the concrete. The shores were probably being removed in some portions of the building during the time preceding the collapse.

The Bridgman Bros. building in Philadelphia was partly wrecked on July 9, 1907, when some foreign laborers removed all the shores under the roof which at that time was only $5\frac{1}{2}$ days old, owing to a misunderstanding of orders. The falling portions of the roof carried with it all the floors directly below, except a portion of the first floor, which partly withstood the shock of the falling concrete.

Failure under test load took place in the roof of the reservoir, at the United States Naval Academy, Annapolis, Maryland, in December, 1908. The footings apparently had been put in on very wet clay, and these footings were without reinforcement except for the wirecloth which was used to reinforce the floor, and which was run into the footings, being depressed under the columns to near the bottom of the footings as well as possible.

On April 7, 1910, a car barn then nearly completed, and belonging to the Shore Line Electric Company, at Saybrook, Connecticut, partly collapsed owing to the premature removal of the forms under the roof.

Finally, the Henke Building in Cleveland, Ohio, was entirely destroyed by collapse on November 22, 1910. (Figure 147). The building was four stories high, and, with the exception of a few of the old brickwalls used for the outside, the entire building fell so as to fill the basement level with the sidewalk of the street. The roof was just completed, and it has been suggested that work was going on in the building on the removal of the few remaining shores in the second story from the top; there were several indications of column failures in that story.[1]

FIGURE 147. WRECK OF THE HENKE BUILDING IN CLEVELAND.
Photo by Alexis Saurbrey, who examined ruins for owner.

In nearly every one of these cases, serious errors in regard to supervision and workmanship have been proved, but it has

[1] While this was in the press, the current issues of engineering papers reported the failure, on Dec. 6, 1911, of a three-story building under erection for the Prest-O-Lite Company, at Indianapolis, with considerable loss of life. The building was of the beam and girderless type, but the details are not available.

not been possible, as far as known, to connect the neglect with the actual causes of the collapse. Nearly all of these buildings fell in the spring or in the fall, when the setting of the concrete is greatly retarded by the cold weather, and even if the days may be quite warm, the nights are cool, and the water used for the concrete very likely quite cold. It is easy to say that if the shores had been left in a few weeks longer, the failures would not have occurred, but it is no easy matter to prove such assertions.

Attention is called to the circumstance that the failures have frequently suggested weakness in certain columns, and in all such cases, the horizontal column reinforcement has been found grossly inadequate or even missing. It is a positive necessity to provide proper ties or hoops in the columns, not one or two column diameters apart, but two to three inches apart, thoroughly binding the loose ends of the hoops together so that they cannot slip. In addition, no shores should be removed before the column sides have been opened and a careful and thorough inspection of all the columns made; not of a few isolated spots on a column here and there, but of the entire height of all four sides of each column.

Undoubtedly, there are grades of efficiency in concrete work as elsewhere, although the best is none too good in most cases. It is however confidently believed that serious failures of reinforced concrete buildings will not occur, if the following simple precautions are taken:

Tie all steel bars into the next span. Use closely spaced hoops in all columns. And see that the concrete is hard before the shores are removed.

CHAPTER XVII

SUPERINTENDENT'S SPECIFICATIONS

The following instructions have been used by the Ransome & Smith Company, as a standard of daily practice for their Superintendents.

General. Order and close attention to details is essential. Want of due care in proportioning, in mixing, or in the placing of the steel may lead to destructive results. Reinforced concrete construction requires close, continuous, intelligent supervision. If this is not given, disaster is not far off. A superintendent places a severe handicap upon himself unless he so organizes his men that from the lowest up to the highest each clearly understands his duties and limitations and knows what he has to do, and unless he so arranges his own time that he can, as a usual thing, devote sufficient time to the unexpected demands that will frequently be made upon him for his attention.

All accounts must be kept up to date and promptly passed upon. All orders must pass through the New York Office except in emergencies, then use emergency orders and forward copies to the New York Office for confirmation.

Temporary Offices and Buildings, Setting up Plant, etc. In setting up the plant see that the mixer is in good line and securely placed upon a level bed. Keep all running gear and wearing parts free from dirt and well oiled and greased. Keep both inside and outside of the mixer, hoisting tub, hoppers, gates, barrows, etc., free from accumulated dirt or concrete of over a day old. Thoroughly cleanse off every night the day's accumulation of concrete and dirt upon tools and machinery. Protect scaffolds and all openings in floors with suitable hand-rails and use every reasonable precaution against accident.

Excavating and Grading. Make these of the dimensions and depth that shall be determined, upon the final examination of the ground. In excavating, give sufficient slope to the sides of the hole or trench to prevent caving in, or protect with sheet piling, and excavate the final depth, corresponding to the depth of the footing, of the exact size required. Do not excavate this final depth much before the time for filling in the concrete. Re-fill as rapidly as the work permits and thoroughly compact all re-filling that subsequently becomes floor-bearing. In grading, follow specifications of the Contract.

Molds. Molds shall be made in strict accordance with drawings, which will be furnished from headquarters.

All molds to be thoroughly fastened together. They must not only be set true and plumb to line, but must be so rigidly held in place that they will resist successfully all tendency to move them that the placing of the concrete may give. All interior, or molding, faces to be thoroughly greased with crude oil before using, and thoroughly cleaned and re-greased at every re-use. All open joints, broken off corners, knot holes, to be properly puttied up with ordinary or improved putty immediately before placing the concrete.

Concrete. Every car load of cement must be tested.

Aggregates will be finally determined upon, at which time the proportions of cement with these will be given. Salt shall be used at the rate of four pounds to a barrel of cement. It shall first be dissolved (in a tank placed above the level of the top of the mixer) to a saturated solution. Then for every bag of cement used in the batch, add three pints of this saturated solution.

In mixing, put the water in first, then the rock, then cement and sand; mix thoroughly and in placing see that the mixed concrete is of such consistency and character that it will pour from the wheelbarrows.

In starting the piers, use a very wet concrete, into each batch of which an additional bag of cement has been placed, for the first foot of height of the piers. Fill each pier in a continuous operation until it is full; short intermissions of time not sufficient to permit the concrete to stiffen may be disregarded and considered as continuous filling. Keep the column work at least twelve hours ahead of the floor work.

All floors must be thoroughly rolled with the first, second, and third roller, beginning with the lightest; continue rolling until the effect thereof is not apparent.[1] The concrete shall be completed in any one unit part before the initial set appears on its surface.

In concreting strike the bars in preference to the concrete between the bars, with the tampers. Great care must be taken to see that the bars are thoroughly embedded in the concrete. Wherever there is a nest of cross-bars that the concrete will not readily penetrate, pour into same sufficient cement grout 1:2 to thoroughly fill all spaces. Special care must be taken (especially in hot weather) to follow up this grout with the body of the concrete before the grout has stiffened. If the circumstances are such that the grout stiffens too quickly for convenient working, time may be gained by throwing on the face of the grout sufficient fresh concrete to cover it, and in turn should this fresh concrete stiffen before it is covered with the main body of concrete, it may be renewed from time to time as above by further small additions of concrete. It is, however, important that neither the original surface nor any of the renewed surfaces be allowed to stiffen before the next layer is applied.

The natural slope of the concrete may be used to terminate any days' work or the work of any period provided the following precautions are taken:—

The surface of this slope must be finished with a drier mixture than usual into which an extra batch of cement has been added. Care must be taken also that this sloping surface is thoroughly tamped down into a compact surface, no loose porous lumps or portions being left anywhere. Before starting the work anew, if this concrete is sufficiently soft to permit of the cement on its surface being thoroughly brushed off with wire brushes, brush it off thus and top off the surface with a liberal coat of pure cement grout well brushed in. If it is too hard for this operation use acid joint.[2] For the concrete needed to cover the sloping surfaces of the previous work throw into each batch an

[1] The use of rollers on concrete floors is not in accordance with usual or current practice. However, it might well be used with beneficial results as shown by my own practice of many years. Note however the necessity of good strong centering that will not yield the least under the heaviest roller used. — E. L. R.

[2] In more recent practice, the vertical joint has been used, as the sloping joint is rather difficult to make and not so easily repaired in case of trouble. — A. S.

extra bag of cement, then proceed with the work as previously described.

Care must be taken to leave the surface of the concrete at the proper level. A variation of more than 1/4″ in the finished level will not be considered as good work.

The concrete must be kept wet for at least ten days. During concreting, a surveyor must be kept constantly at the work to determine whether or not there is any settlement in the falsework, and, in case there should be in exceptional cases, the defect should be rectified before the concrete sets. This also applies to the alignment of the exterior surfaces of the work.

Steel. All steel shall be kept as free from rust as practicable. All bars must be placed as shown on the drawings. No variation in height of over 1/2″ is allowable, or in other dimensions of over 3/4″. No steel must appear on the surface of the work. Steel that would otherwise reach the surface must be wrapped with one or more turns of protected wire or stout marlin.

All the steel must be placed ahead of the concrete except where instructions are given to the contrary (in very exceptional cases only).

Finishing. All floors shall be treated with acid joint and finish, except where the finish is put on before the floor is thoroughly set. This latter shall be of the proportion given, mixed quite stiff and thoroughly well troweled down and worked to a true smooth finish. Extreme care must be taken to follow closely the instructions given here below relative to the acid joint.[1]

Acid Joint. (1) Thoroughly sweep the floor, removing all loose concrete dust and débris, etc.

(2) Wash floor thoroughly with water.

(3) Wash floor with acid mixture (1 acid 18% to 1 water) pouring it on the floor freely and slowly sweeping it forward. Follow this washing with a second and third in like manner.

(4) Give the floor a final and *thorough* washing of water. Immediately before laying the finish:—

(5) Thoroughly wet the floor.

[1] This method is covered by my U. S. Patent No. 860,942, Oct. 3, 1905. — Ernest L. Ransome.

(6) Rub in a pure cement cream with wire brushes, sweeping forth and back, going over the same ground seven times.

(7) Before this shows signs of setting, sweep over it more of the cement cream so as to leave on the surface a thickness of about 1/8″. This cream should be thicker than the first.

(8) Before the above layer shows signs of setting, put on the finish.

CHAPTER XVIII

THE ENGINEER

As compared with other methods of construction, reinforced concrete is essentially a manufacture. From the earliest days of the art, this was recognized by the makers, who called themselves artificial stone manufacturers and concrete manufacturers. The contractor receives the raw materials in the form of cement, sand, stone, and steel bars, and from these he manufactures the structure, while in other types of building contracting, the finished product is received at the building, and then simply erected in place. Hence the reinforced concrete contractor is charged with two duties, namely, manufacture and erection, where the other contractor has only one, namely, erection.

It follows that expert knowledge, similar to that possessed, for instance, by the steel mill organization, must in some manner be supplied on the reinforced concrete job. According to circumstances, the expert services are provided by either the owner, the architect, the contractor, or the local building department, if indeed they are not wholly absent, which appears to happen occasionally. The latter case is entirely too frequent, due to the prevailing lack of understanding of the difficulties incidental to reinforced concrete work. It is the duty of those who know, to emphasize this fact, each in his own locality, so that the general public may at last appreciate the absolute necessity of expert skill on all reinforced concrete work.

It is not believed that building ordinances or regulations can cope with this problem successfully. In Cleveland, Ohio, the owner is required by law to provide an inspector who shall be present at all times when concrete is being placed on reinforced concrete buildings; the inspector must pass an examination before the building authorities. But this examination is so elementary that nothing even remotely approaching expert supervision is obtained. In many respects, the ordinance is objectionable to the owner, who cannot always command the services of an examined inspector at the proper moment, as well as to the contractor, who may sometimes have to wait for the inspector. In spite of these minor objections, the system is undoubtedly beneficial in Cleveland at the present time, although the possibilities for misuse are great and always present. The chief objection would seem to be in the fact that the owner's conscience is lulled to sleep in the hope that a paternal city department will see him through all troubles, while as a matter of fact the inspection is barely sufficient to guard against gross and continuous blunders.

In Boston, Mass.,[1] the law provides: "When the structural use of concrete is proposed, a specification stating the quality and proportions of materials and the methods of mixing the same shall be submitted to the Building Commissioner, who may issue a permit at his discretion and under such further conditions in addition to those stated below as he sees fit to impose." The "conditions stated below" give the allowable unit stresses and other provisions foreign to our present purpose; the discretionary conditions which the Commissioner imposes at the present time are:

(1) That the plans before being submitted to him shall have been approved by an expert engineer satisfactory to himself.

(2) That during the placing of concrete an inspector shall be employed at the expense of the owner; the inspector must be satisfactory to the Commissioner, and must report to the Department of Buildings.

In regard to the expert engineers, the Commissioner reserves to himself the right to pass upon them at any time. Objections have been raised to this arrangement on the ground that the expense of examining the plans should be borne by the Building Department, and not by the owner (although it seems proper that each owner should pay the expenses of his own plans). The advisability of employing an expert in the department has been considered, but so far without result.

In regard to the compulsory inspection, the same objections may be raised as in Cleveland, that really efficient inspection is not obtained in that manner, and that the owner meanwhile is brought to believe that his work is efficiently inspected.

[1] The authors are indebted to Mr. J. R. Worcester, M. Am. Soc. C. E., for information in regard to the Building Regulations in force in Boston.

The Building Regulations of Boston and Cleveland, just cited, throw a very remarkable light upon prevailing conditions. It is almost unbelievable that it should be necessary to actually force the owner into engaging adequately trained men to plan and supervise the structure in which the owner, more than any one else, is vitally interested. Undoubtedly, the efforts of local building departments have succeeded in keeping the standards of workmanship and design above a certain level, even if that be low, but it must not be forgotten that the final decision rests with the public generally and the building owners in particular, and for that reason, the real problem before the concrete engineer today is to reach and educate the public so that better work is not only insisted upon but also paid for.

It would be very desirable if uniform regulations could be made for methods of design and calculation, eventually in the form of State Laws. Efforts toward standardization of calculations have been made by the American Society of Civil Engineers and others, and that such recommendations or regulations are not impractical may be seen from their successful operation in Prussia, Austria, France, etc. Owing, however, to the great variation in available supplies of aggregate, the *allowable stresses* must always remain a local issue.

Various influences are at work which greatly retard the development of sound engineering. Certain concerns engaged in the selling of reinforcement will furnish free plans showing designs calculated to land the job rather than to give efficient service. The method is objectionable when worked through the medium of a small contractor, but much more so when a so-called "architect" is made to act as a cat's-paw. The architect (or engineer) who holds himself out as qualified to design reinforced concrete work, and either has not, or does not provide for the requisite skill, is guilty of deception, and obtains his money under false pretenses. As a matter of fact, all our best architects have competent engineers on their staff, or engage the necessary talent when required, and the owner can always obtain such services by simply insisting upon having them.

Another objectionable practice has sprung from the indiscriminate use of "tables of design." The modern steel industry would certainly be an impossibility without standard shapes, and here the structural steel tables in common use are the only proper thing. But while a certain degree of standardization in reinforced concrete construction is urgently important, it is practically impossible to provide for the innumerable possibilities of design, at least at present. Moreover, while the table itself may be a labor-saving device, it is likely to be used most by those who are least conversant with the underlying principles, leading to disastrous results.

Again in certain sections, particularly in the Middle West, a class of contractors has been created whose slogan appears to be: "get the job at any cost." No contractor can afford in a lump sum contract to take work at less than actual cost to him plus a reasonable profit, the cost to include overhead expenses, depreciation, idleness of plant and staff, contingencies, etc. For a while, a contracting business may be run in violation of these principles, but not for long. Of course, every job on which the specifications are honestly and consistently enforced hastens the day of judgment for such concerns, and they strongly resent anything that looks like supervision. These concerns have injured not only themselves, but have succeeded in lowering the general standard of workmanship by training foremen and young engineers in sloppy and slovenly work. When these abuses become too great emergency provisions are in order, and the compulsory inspection paid for by the owner under the supervision of the building department is one way.

At the present time, there are reasons for believing that reinforced concrete contracts should be let on the "cost plus profit" basis. Such contracts protect the owner against pooled bids and against extortionate charges for contingencies or profits. It is evident that the structural steel contractor has no "contingencies of manufacture," but only "contingencies of erection," while the reinforced concrete contractor has both.

In fact, if the contract be not awarded to the lowest bidder, there is no good reason for taking bids, and if the owner has so much confidence in any one bidder that he prefers him in spite of his higher bid, he might as well trust him to the extent of giving him the contract on the cost plus profit basis. We are not here concerned in discussing the various types of contracts possible under this system, as to whether the maximum cost ought to be guaranteed or not, or whether the profit should be a percentage of the actual cost or a certain stipulated sum.

It is believed that such contracts are usually given to contractors having an engineering department in their organization, and who are, as a matter of fact, "contracting engineers" whether so called or not. We must remember that reinforced concrete construction was first introduced, and has since mainly been developed, by just such men or concerns. It is safe to say that a very large portion, if not the largest portion, of all reinforced concrete buildings of any consequence is erected by "constructing engineers," who plan, design, and erect the work from start to finish, frequently on the cost plus profit basis. The owner should have these plans checked by a consulting engineer and provide for adequate inspection of the work. The position of the inspecting engineer is one that calls for considerable tact, because he, as well as the contractor, are virtually members of the same organization, viz., of the owners' building staff.

Under the lump sum contract, the engineer's position is radically different. He, and he alone, should prepare the general and detail plans, with adequate specifications, and once the contract is let, it becomes his duty to *enforce* the specifications in letter and spirit, making himself as disagreeable as conditions demand. Even if the specifications (or contract) give the engineer the right to make necessary alterations, he should be exceedingly careful not to waive any of the requirements by commission or omission. Inspection of this kind is efficient only when explicit and full specifications have been prepared, but this does not mean that the specifications should be burdensome or unfair to the contractor. It is no easy matter to write good specifications for reinforced concrete work, and it requires first of all full acquaintance with local conditions. There are many places, for instance, where "clean," "sharp" sand cannot be obtained locally, and the engineer must so word his specifications that suitable sand is called for, and he must then see that good sand is really used; — not, as some engineers do, call for clean, sharp sand, and then allow the use of sand that is neither the one nor the other.

Attention is called to the difficulties encountered in making monthly estimates for reinforced concrete buildings. The falsework enters only as machinery, tools, or other appliances, and its full value should not at any time enter into the estimate, but only a certain proportion of its value. This leaves considerable room for argument as to just what proportion to include; the better and safer way is to have a clause in the contract stating that a certain reasonable proportional amount of the contract price must be paid: (1) when the footings are in; (2) when the first floor has been concreted, etc., etc. It is very much easier to arrange the amounts to be paid before the contract is signed than after the work is under way.

It is now evident that whatever the position of the engineer, — whether connected with owner, architect, or contractor, — he must possess certain qualifications of his own. First of all, he must make himself felt as a useful factor in the community, and not be satisfied with remaining a subordinate, and apparently superfluous appendix. He alone has it in his hands to make the industry advance or decline, and his essential function is, not only to economize in the proper place, but to make the owners see the folly of parsimony. He will have to overcome criticisms of impracticability and extravagance, and this will be the more difficult as he will rarely be brought face to face with the charges.

He must be an expert designer, not only of the usual ribbed floors, arranged in the conventional cigar-box type of factory building, but also of the more complicated types of flat floors, of ribbed arches and other unusual forms for which reinforced concrete is so well adapted and as yet so little used. Nevertheless his ability as a mathematician must not kill his ability as a business man, for if he cannot get the work to exercise his mathematics on, he will have scant use for them. He must be fully posted on methods of erection — not only how to do things, but also how not to do them — yet his knowledge must not make him overbearing with the common foreman who "knows everything about it," yet whose main asset is his ignorance.

Granting now that our engineer approaches to some extent the ideal just outlined, he must also possess a certain amount of skepticism in regard to precedents. Without question, there are wide fields for investigation as yet open. We have referred in an earlier chapter to the fallacy of too implicit faith in cement testing. We have considered the impossibility of current ideas of shear in reinforced concrete beams. There may be, and probably are, many others. Criticism of this kind is beneficial, not only professionally, but sometimes financially as well, because sound criticism leads to improvements, and good improvements are well worth while.

In order to gain material benefit from an improvement or invention, it must be patented. Reinforced concrete men are too prone to decry the value of patents generally, but this attitude appears to be founded in ignorance. In order to avoid infringement, the engineer must certainly be familiar with patents and patent law; only in that way he can save the client undue expense and trouble, and judge for himself of the value of a new invention. The only circumstance saving many a man from patent suits is that the patentee cannot afford the expenses of court trial, which may run anywhere from $5,000 to $20,000 or more, and extend over many years. If there are any good and substantial reasons for granting patents, the engineering profession should recognize the existing conditions and inform themselves, treating patent rights in the same manner as they do other property; if no such reasons exist, the engineers should use their influence in having the patent office abolished. There is little likelihood that the latter alternative will be followed, and patents should therefore be respected. One way of ensuring the rights of the patentee would be to have an injunction issued at once when proper evidence was presented to the court, and leave it for the infringer to prove the patent invalid; as it is, the patentee practically has to prove the validity of his patent before any injunction will be issued. It would well pay the owner to see that his engineer is well posted on the question of patent rights, for if infringement should occur, the patentee will certainly look to the owner for reparation.

CHAPTER XIX

THE THEORY OF BEAMS AS ILLUSTRATED BY TESTS

The Extensibility of Concrete is not changed by the presence of reinforcement. It was discovered in tests made at the University of Wisconsin in 1901–1903 that beams cured in water and partially dried showed "watermarks" or fine dark lines on the tension side under loads which would have fractured non-reinforced pieces, and it was proved that these watermarks indicate cracks.[1]

In reinforced concrete beams, these cracks appear under tension stresses in the steel of about 5,000 lbs. per square inch, and we are therefore not justified in calculating on any tensile resistance in the concrete.

The Shear Resistance of Concrete is not affected by the presence of reinforcement. Prof. Mörsch[2] made shear experiments with cement mortar prisms 7″ x 7″ in section and found:

for mixture 1:3; 2 years old:	879,835,	1098 lbs./sq. inch
	average	937 lbs./sq. inch
for mixture 1:4; 6 weeks old:	549,593,	441 lbs./sq. inch
	average	528 lbs./sq. inch

Reinforced prisms of same mixture, size, and age as the last series sheared under the following stresses: 550, 495, 528, 451, 473 lbs. per square inch. It made little difference whether the reinforcement was straight or bent. The final carrying capacity of the reinforced prisms was, however, much greater than their apparent shear resistance, for after the concrete had sheared it was still possible to increase the loads considerably. Professor Mörsch considers that this increase was due to the shear resistance of the steel, which, mathematically speaking, was stressed in shear as follows, when the final collapse took place:

[1] Turneaure and Maurer: Principles of Reinforced Concrete Construction, 2d Edition, p. 42.

Subsequent tests by Bach (Zeitschrift des Vereines deutscher Ingenieure, Band 51, Nr. 26) have fully supported the Wisconsin tests.

[2] Mörsch: Concrete Steel Construction, p. 33 ff.

Specimen 1:	47,650	lbs./sq. in.	straight bars only
" 2:	45,230	" " "	" " "
" 3:	55,050	" " "	straight and bent bars
" 4:	47,080	" " "	" " " "
" 5:	50,350	" " "	" " " "

The ultimate shear strength of the steel was only 47,790 lbs. per square inch, so where specimen 3 acquired its additional 17 per cent. strength does not seem clear. Specimen 2 failed under a load of 40 tons, "at which point a horizontal crack appeared at the left end." This, we know, is an indication that the steel is pulling out of the concrete, and it seems altogether likely that the resistance really measured in these specimens was the tensile resistance of the reinforcement, in accordance with the theories advanced in Part II, Art. 57, of this book.

Various other tests have been made to determine the resistance of concrete to pure shear. They generally confirm the figures given directly above, but the results vary greatly owing to the great difficulty in eliminating tensional stresses. In practical construction, pure shear is rarely encountered in reinforced concrete beams.

The Function of the U–Bars. With the foregoing remarks in mind we must admit that the U-bars cannot in any way influence the shear resistance of the concrete. If we consider the U-bars as active in shear, their action cannot take place before the shear resistance of the concrete is exhausted, and whatever view we take of the stresses, the total shear resistance of the beam is not the sum of that of the concrete, and that of the U-bars (or other "shear" reinforcement). In this book, the U-bars have been considered as (1) retarding the sliding of the main tension reinforcement and (2) supplying the vertical tension resistance caused by deviation from the equilibrium curve of either the compression or the tension "chords." The first proposition is easily investigated by test; the second is closely related to problems connected with trussed rods and kindred matters, and will be considered in that connection here below.

The Stirrups Retard the Sliding of the main tension rods. The "Commission du Ciment Armé" (1907) tested specimens as shown in Figure 148 *a*, *b*, and *c*. The specimens gave the following average sliding resistance per sq. inch of embedded surface, the first group having stirrups of flat iron, the second of round iron:

	6 months old	3 months old
a	159 lb./sq. inch	125 lb./sq. inch
b	214 lb./sq. inch	252 lb./sq. inch
c	281 lb./sq. inch	284 lb./sq. inch

or about the same values for the specimens with stirrups as for rods centrally embedded in a block of concrete. This shows the increasing importance of U-bars in beams with thin concrete covering on the rods, even in the case where U-bars are not theoretically required.

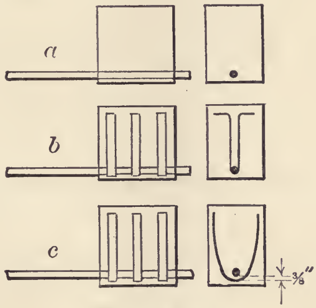


FIGURE 148.

These tests show the gripping action exerted by the U-bar on the rod, and explain in part the tendency of the beams to crack at the U-bars, because the U-bars act as washers on the rod, so that the concrete naturally would split immediately behind such points.

Straight Reinforcement in T-Beams — German Tests. In the famous series of T-beams tested by Prof. Mörsch, beams I, II, and III were equipped with U-bars for one-half the length only. The beams all failed at the end without U-bars under loads fairly proportional with the width of the stem, showing that the resistance of the stem was the deciding factor. It makes little difference for our present purpose whether the failure was caused by actual shear or by the pulling out of the reinforcement; — Prof. Mörsch appears to favor the former of these alternatives, although in the two beams with narrow stems, the concrete surrounding the reinforcement was split off, indicating that the concrete was not sufficient to resist the lateral expansion, thus allowing the rods to slip. The resistance called into action in this manner would be proportional with the thickness of the stem.

These tests show conclusively that T-beams with straight reinforcement only and without U-bars are not economical structures. As to the U-bars themselves, the tests show they are beneficial, and Prof. Mörsch further states:

"If the cause and the formation of the cracks in these three beams are examined, it is established that the cracks first became visible where the moment was greatest, and that with increase of load more distant cracks appeared. On the end supplied with stirrups, the cracks appeared to occur at the sections in which the stirrups were located, since the concrete section was weakened at those points." The same observation has been made in other investigations.

These beams were tested with a uniformly distributed load covering the entire span.

Bent Reinforcement in T-Beams—German Tests. In continuation of the tests just described, Prof. Mörsch investigated

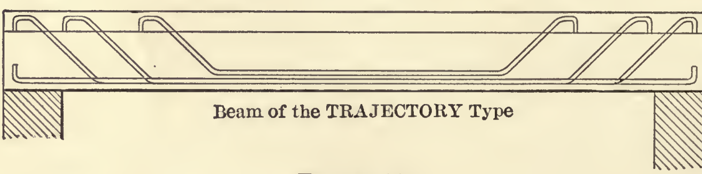


FIGURE 149.

several beams with a combination of bent and straight bars. Two distinct types were used, the bent bars being of either the "trajectory" type (Figure 149) or of the "suspension" type (Figure 150). In the table herewith, the principal details of

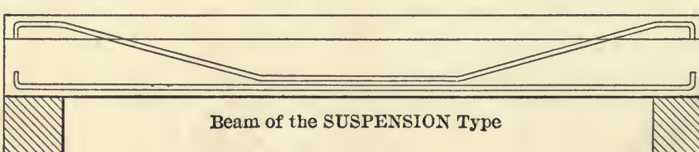


FIGURE 150.

the arrangements are given (the letters *T* and *S* indicating the type of bent reinforcement), as well as the ultimate load.

TABULATED RESULTS OF PROF. MÖRSCH'S BEAM TESTS

Beam	Type	Tension Rods: Bent No.	Tension Rods: Bent Diam. in m/m	Tension Rods: Straight No.	Tension Rods: Straight Diam. in m/m	Thickness of Stem c/m	U-bars	Type of Loading	Ultimate Load in metric tonnes
IV	T	3	15 and 1		18	14	none	Uniform	42.0
VI	T	3	15 and 1		18	14	full supply	load	37.8
V	S	2	15 and 2		16	14	one end only	covering entire span	31.0
								Two	
VII	T	3	16 and 1		16	14	full supply	concentrated	34.0
VIII	S	2	16 and 2		16	10	one end only	loads	23.4
IX	S	2	16 and 2		16	14	one end only	at third points	25.6
X	T	3	16 and 1		16	14	one end only	One	27.0
XI	S	2	16 and 2		16	14	none	concentrated	26.0
XII	T	3	16 and 1		16	14	none	load at center	26.0

The tests naturally divide themselves into three groups, according to the manner of loading:

(1) Uniform load, beams IV, VI, and V. It is at once apparent, by comparing beam IV (without U-bars) with beam VI (with U-bars), that the influence of the U-bars is very slight, if any, the difference in ultimate load being accounted for by the fact that the ends of the straight bar in beam IV were hooked, while those in beam VI had no hooks. Both of these beams were of the trajectory type, and if we compare them with beam V of the suspension type, the superiority of the trajectory type seems clearly established. But we must not lose sight of the fact that in the two first beams three of the four rods were bent up, while in the latter, only two of the four rods were bent up. This beam failed in the end without U-bars, and while therefore this group does not prove the author's theories, as outlined in a preceding chapter, it does not disprove them, and still leaves the question open whether or not the bending of one additional rod, or the proper use of U-bars, would not have changed the results materially. It will be remembered that in a T-beam, the straight reinforcement is effective only as reinforcement of the stem, while the bent bars correspond to the flange; if the rods are not so arranged, stirrups must be introduced to again bal-

ance the design, the size of the U-bars being in direct ratio to the violation of the principle outlined.

(2) Two concentrated loads, beams VII, VIII, and IX.

Here again, beam VII of the trajectory type, with a full supply of U-bars, is compared with two beams of the suspension type, the two latter being without U-bars. Again the trajectory type seems superior to the suspension type, and again we find the reason to be that in the trajectory beam, the proper amount of rods have been bent up, while in the suspension type, only two of the four rods have been bent, and no U-bars have been introduced to overcome the deficiency.

It is rather interesting to note that the difference in width of stem between beam VIII (10 cm.) and beam IX (14 cm.) affects the ultimate strength but slightly, both beams being of the suspension type.

In his discussion of these tests, Prof. Mörsch has taken occasion to criticize the suspension, or Hennebique, type. It is to be regretted that the tests were not carried out so as to have the same number of bent-up bars in both of the types considered, in which case the suspension type would probably have stood up as well as the trajectory beams. It is only fair to note that the U-bars or stirrups have always been considered as an essential part of the Hennebique system, and that such tests as these, however valuable otherwise, give no indication whatever as to the merits of this system.

(3) One concentrated load at center, beams X, XI, XII.

In this group, the two systems give the same carrying capacity, owing undoubtedly to the fact that in no one of these beams the reinforcement is arranged according to the equilibrium curve, while in no case U-bars have been introduced to compensate for the deviation.

Bent Bars in T–Sections—Author's Tests. The beam tests just referred to were published by Prof. Mörsch in "Deutsche Bauzeitung," April 13, 1907. It occurred to the author of the theory of this present volume that the description of the action of the suspension rods was subject to doubt, for the reasons outlined above, and that additional information might possibly be gained by tests on beams with trussed rods only. The author designed a series of nine test beams which were tested in the winter 1907–1908 at Case School in Cleveland, in co-operation

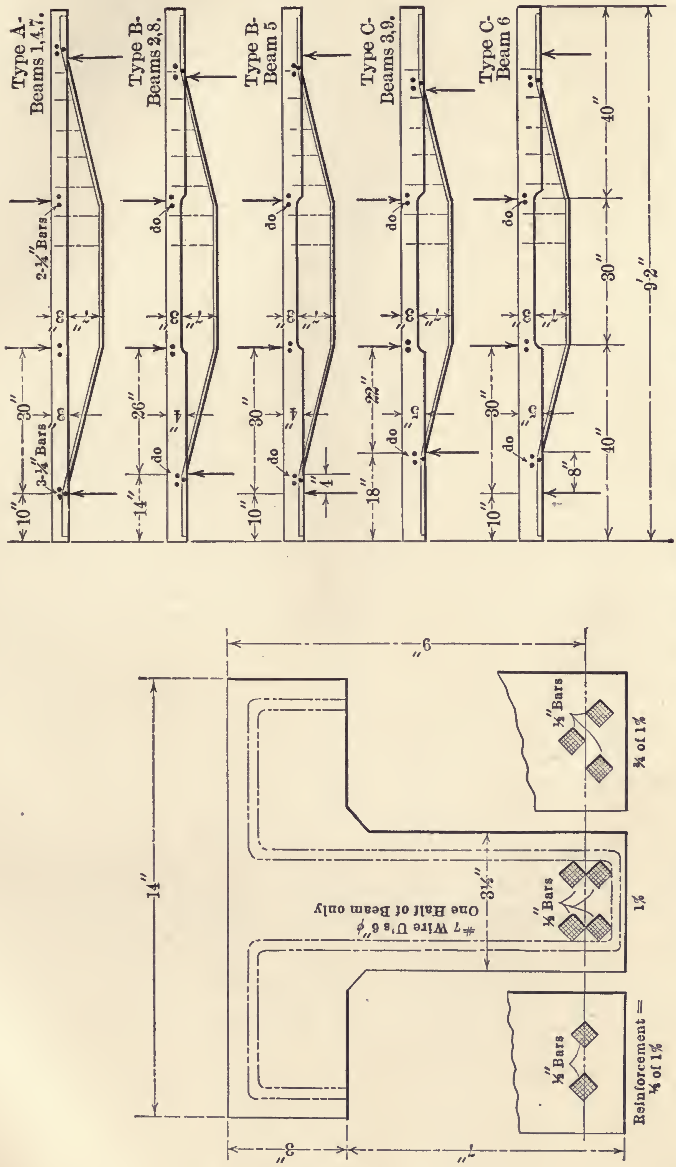


FIGURE 151.—DIAGRAM OF BEAMS TESTED AT CASE SCHOOL

with Prof. F. H. Neff. It will be seen from Figure 151 that these beams had no straight reinforcement, and that the sloping stem terminated at the supports, so as to make the system one of equilibrium under two concentrated loads. The results were first published in the Engineering Record, August 22, 1908, from which the following is an extract:

"Three different molds were made, types A, B, and C, respectively, each one of which was used three times with a different percentage of steel for reinforcement of the beam. In this way, three beams of type A were made, one of which was reinforced with 0.5 per cent., one with 0.75 per cent., and one with 1.0 per cent. In the same way three beams B and three beams C were made, reinforced as described, so that of the total number of nine beams no two were alike in all respects, but any one beam would have a corresponding one which was different in one detail only. In this way, it would be possible to compare the beams and find the exact effect of a certain change, which is a safer way than to try to obtain absolute results from so few tests.

"All the beam swere provided with U-bars in one end only, the object being to show that the stirrups were of no consequence at all. The stirrups made no difference in the results obtained, four of the nine beams failing in the end equipped with U-bars.

"Two short cross bars were placed in the slab at the points where the loads were applied, and three similar bars were placed in the slab near the support. These bars were ¼-inch square twisted bars. The main tension bars were ½-inch square twisted Ransome bars. It was found that the elastic limit of these bars averaged about 56,000 lbs. per square inch, and their ultimate breaking strength was 73,600 lbs. per square inch.

"The concrete was made quite wet and very carefully placed. The mixture used was 1:2:3½, Lake Erie sand and Euclid bluestone being used for the aggregates. The strength of the cubes was low, as might be expected with the aggregates used, and the average of the 6-inch cubes in pounds per square inch was as follows:

Age, days	7	14	28	60
Strength, pounds	660	1,065	1,440	1,787

"The beams were all tested when sixty days old. In the table here below the results are given, and this table, together with the diagrams of the beams, should give all the information needed. Attention is called to the ways of supporting beams 5 and 6. While all the other beams are supported at the point where the sloping stem begins, these two beams are supported further out from the stem, making the overhang shorter for them than for the similar beams of same type.

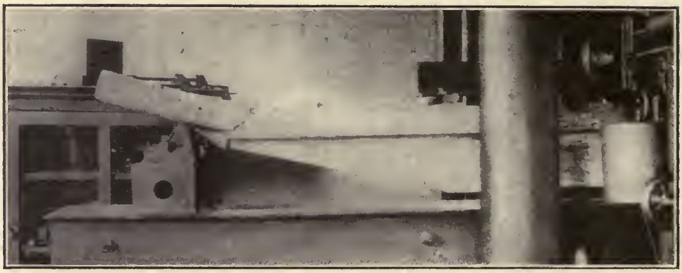
"As to the column headings used in the table, the percentage of reinforcement is calculated with reference to the 'enclosing rectangle' proposed by Professor Talbot. Under 'lever' the distance from point of support to point of application of the load is given, while 'overhang' means the length of the projecting end beyond the support.

"The bending moment given in this table is found by multiplying the 'lever' by one-half of the ultimate load, disregarding entirely the weight of the beam itself. The lever arm of the internal stresses is assumed to be 0.85 times the distance from the top fiber to the center of the steel, which distance is approximately 9 in., giving a lever arm of 7.65 in. This, of course, is not quite correct, as the position of the neutral axis varies with the percentage of steel and the coefficient of elasticity, which latter again depends upon the stress on the concrete. It is, however, sufficiently accurate considering the unavoidable variations in the position of the steel bars and in the elastic properties of the concrete, and the 'total stress in the steel' may therefore be found by dividing the bending moment by 7.65, giving the values shown in the table as well as the stress in the steel per square inch of its cross-section.

Results of Tests at Case School.

Beam.	Type.	Per cent.	Lever.	Overhang.	Ultimate load.	Bending moment.	Stress in steel	
							Total.	Per sq. in.
1	A	0.5	30 in.	10 in.	12,500	187,500	24,500	49,000
2	B	0.5	26 in.	14 in.	16,000	208,000	27,200	54,400
3	C	0.5	22 in.	18 in.	27,800	305,800	39,900	79,800
4	A	0.75	30 in.	10 in.	11,900	178,500	23,300	31,000
5	B	0.75	30 in.	10 in.	16,200	243,000	31,800	42,400
6	C	0.75	30 in.	10 in.	16,000	240,000	31,400	41,800
7	A	1.00	30 in.	10 in.	13,950	209,250	27,300	27,300
8	B	1.00	26 in.	14 in.	22,000	286,000	37,400	37,400
9	C	1.00	22 in.	18 in.	28,900	317,900	41,600	41,600

"Referring now to the several photographs of the beams after failure, it will be noticed that the failures are of uniform nature. Comparing the figures given in the table above, it will be seen that the ultimate load varies greatly as well as the total stress and the stress per square inch. If the failure has



Beam 1 FIGURE 152.

Beam 2 FIGURE 153.

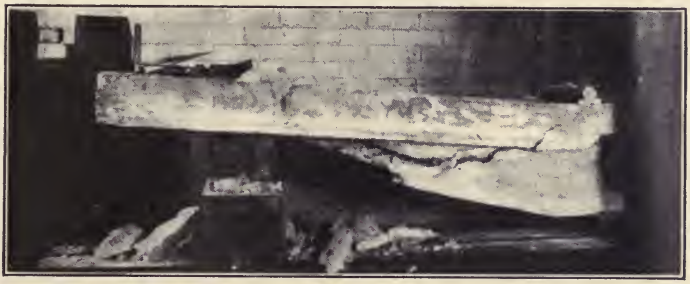
Beam 4 FIGURE 154.

Beam 5 FIGURE 155.

THE CASE SCHOOL BEAMS AFTER TESTING.

a common cause in all these beams it cannot be due to either tension or compression in the usual sense of the word. It may also be assumed that shear had little to do with the failure.

Beam 6 FIGURE 156.



Beam 8 FIGURE 157.

Beam 9 FIGURE 158.

THE CASE SCHOOL BEAMS AFTER TESTING.

On account of the trussed form of the beams, the steel follows the curve of equilibrium of the external forces acting upon the beam, and the only stresses possible are tension in the steel and compression in the concrete.

"This is also evident from the behavior of the beams under load. The cracks started on the tension side and opened slowly with increasing load, at the same time becoming longer, until finally the compressive area left above the top of the crack became too small to carry the stress on it and crushed. A shear crack cannot grow in this manner. It is well known that the maximum shear stress does not occur at any fiber near the extreme top or bottom of a beam. Therefore, when the crack extends up into the stem and reaches the neutral axis, the shear resistance of the beam is practically exhausted.

" The beams also made it evident in other ways that no vertical shear was active. In some cases the beams had received a vertical crack in handling, the crack being located about 3 in. inside the support, and extending clear through the concrete. At first, it was believed that these beams would not give a fair test, and it was taken under consideration to leave these beams out. It proved, however, that the crack closed up as soon as the load was put on, and after the load was increased to a certain amount, the cracks were hardly visible, while the final failure took place some distance from the injured section. If there had been any vertical shear acting on the beam, the ultimate load would have reached a comparatively small value only, and in all probability the injured section would have sheared off at once.

"The tension in the steel must be constant from end to end of the beam between the supports. The steel would have a tendency to pull out of the overhanging ends with a force equal to the total pull in the steel, which is the same near the supports as at the center of the beam. The overhanging ends furnish the necessary anchorage for the bars on account of the grip of the concrete around the bars, which increases with the compression in the concrete, and, therefore, also with the load, the horizontal cross-bars giving the required horizontal restraint of the concrete to produce the desired effect. The numerical value of the length of the anchorage may therefore be expressed in figures by simply dividing the length of the overhang into the total pull on the steel, the quotient giving the value of the bond in pounds per lineal inch of embedment, regardless of the amount of steel. This figure is given in the accompanying table, the length of the anchorage being the length of the overhang, and disregarding the extra length of the hook at the ends of the bars. Beams 5 and 6 are not included in the table, as these beams had an overhang of only 10 in., leaving a horizontal space inside the support, and this, of course, makes it impossible to compare these two beams directly with the rest.

VALUES OF BOND OBTAINED

Beam	Type	Overhang	Total pull in steel	Bond per lin. in.
1	A	10 in.	24,500	2,450
2	B	14 in.	27,200	1,940
3	C	18 in.	39,900	2,230
4	A	10 in.	23,300	2,330
7	A	10 in.	27,300	2,730
8	B	14 in.	37,400	2,670
9	C	18 in.	41,600	2,310

"This table, it is believed, is remarkable when the uniformity of the results is considered. The beams tested here had reinforcement varying from ½ of 1 per cent. to 1 per cent., spans varying from 74 to 80 in., and tension stresses in steel varying from 27,300 to 79,800 lbs. per square inch. It seems safe to say that these beams all failed by sliding of the steel.

"So far, no attention has been paid to beams 5 and 6. The overhang for these beams was 10 in. in each case, the slab continuing for a distance inside the supports. The bond stress developed in the overhang, if figured as for beams above, becomes 3,180 and 3,140 lbs. per linear inch, or quite high when compared with the results of the table above. Remembering, however, that the straight portion of the bar is continued inside the supports for a distance of 4 and 8 in., respectively, the bond, if distributed over the total distance of 14 in. for No. 5 and 18 in. for No. 6, becomes:

Beam	Type	Overhang	Total stress in steel	Bond per lin. in.
5	B	10″ + 4″ = 14″	31,800	2,270
6	C	10″ + 8″ = 18″	31,400	1,745

"If any importance can be given these two isolated results, they would show that the bond inside the support is quite as effective as that outside the support, but for a short distance only, and that its value decreases rapidly with the distance inside the support."

The lessons to be drawn from these tests are:

(1) That, with the arrangement used, the presence or absence of U-bars does not influence the strength of the beam.

(2) That "shear," properly understood, does not exist in beams of this kind.

(3) That, with proper arrangement of the end supports and of the anchorage, such beams will not fail until the compressive strength of the concrete, or the tensile strength of the steel, is exhausted.

(4) That such beams are rational structures capable of practical and economical use.

(5) That the sliding resistance of the steel does not depend upon the number or size of the individual rods, but only upon the anchorage of the **group of rods**, the length of embedment being much more important than the diameter of either each rod or of the group of rods.

Effect of Joint between Slab and Stem, Tests by Professor Johnson. In connection with the introduction of the Ransome Unit System (p. 162 ff.) in Boston, a series of very interesting tests were made on T-beams of both the monolithic and unit types, reinforced with straight bars only, and with both straight and bent bars. All the beams had U-bars. In the "Unit" beams, the slab was cast from four to nine days later than the stem. A total of twenty-eight beams were prepared, of which eleven have so far been tested, the balance being held for a longer-time test.[1] The beams were all reinforced with Ransome steel, that is, cold-twisted squares. The U-bars were round except in Type C, where square twisted U-bars had been used.

Type A, Beams 1, 2, 4, 5, 9, 10, *and* 12. *See Figure* 159.

[1] The authors are indebted to Prof. L. J. Johnson, M. Am. Soc. C. E., for the following data, and for permission to publish the same. The beams were designed by Prof. Johnson, by Mr. J. R. Worcester, M. Am. Soc. C. E., Consulting Engineer, by Mr. J. R. Nichols, Jun., Am. Soc. C. E., by the Concrete Engineering Co. of Boston, and the Ransome Engineering Co. of New York, each having designed one series of beams or contributed to the design by suggestions. Professor Johnson, who made the tests on the testing machine in the Harvard University laboratory, expects to publish in due season a complete report of both this series and of the long-time tests. The authors of this present volume, eye-witnesses of these tests, are solely responsible for conclusions reached herein.

In the Unit beams, the top of the stem was either left fairly smooth, as it would be in usual every-day practice, or corru-

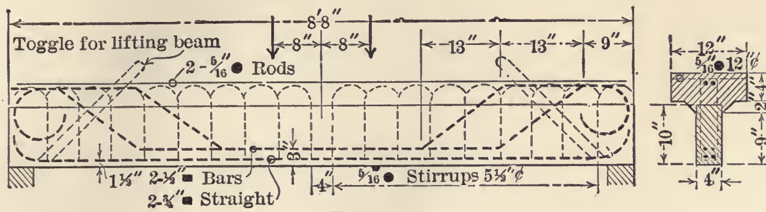


FIGURE 159.

gated as shown in Figure 160. The ends of the stem rested in previously prepared seats (Figure 161), and the joints were

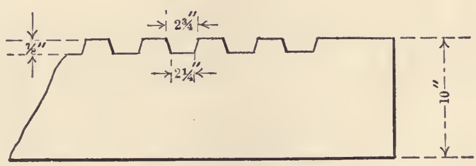


FIGURE 160.

sealed with grout to ensure a similar action as obtained in actual construction, where the Unit beam rests in a pocket in the girder. Nine days after the casting of the stem, the slab was put on, while in the monolithic beam, the entire amount of concrete was, of course, deposited in the forms in one operation.

Beam No. 5 was a Unit beam, with the top of the stem corrugated. The age of the stem was forty-five days, that of the slab thirty-five days. At a total load of 12,000 lbs. the first tension crack appeared near the middle of the span. Inclined cracks became evident near the ends under a load of 23,000 lbs.; the ultimate load was 41,000 lbs., when failure occurred, through compression of the slab between the loads, and slipping of the straight tension bars (see beam No. 12 below).

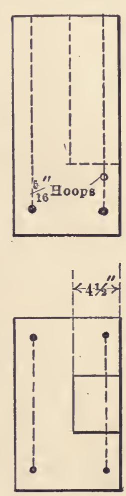


FIGURE 161.

Beam No. 4 was a Unit beam, the top of the stem being fairly smooth; that is, no attempt had been made toward getting a particularly rough surface. The age of the stem was forty-five days, of the slab thirty-six days; the first crack was observed under a load of 20,000 lbs.; ultimate failure took place under 48,000 lbs. in precisely the same manner as in No. 5.

Beam No. 1 was exactly similar, except that slab and stem were both one day older than in No. 4; the ultimate load was 49,400 lbs., and the beam failed in the same manner as the foregoing.

Beam No. 2 was of the same age and detail as No. 1; the first crack was observed at 10,000 lbs. loading; the ultimate failure occurred in the same manner as above under 54,300 lbs. total. The higher load on this beam is perhaps due in some measure to the fact that the rocker-supports for the beam came to a bearing, making possible some horizontal thrust on the beam.

Beam No. 12 was monolithic, forty-one days old; of same design as the foregoing Unit beams, except that the fillet between stem and slab was slightly reduced (see Figure 162). The first

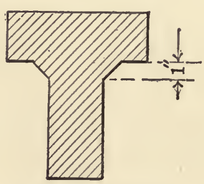


FIGURE 162.

crack occurred at 4,000 lbs., the ultimate load was 45,800 lbs., and failure occurred through a slip of the straight reinforcement, causing the sudden collapse of the left end.

Beam No. 9 was of the same general design, cast in one piece, and forty-one days old. The first crack occurred at 3,000 lbs.; the beam failed suddenly at 50,000 lbs. by slipping of the rods at the right end.

Beam No. 10 was also a monolith forty-one days old, showing a tension crack at 6,000 lbs., with ultimate failure at 52,400 lbs. from a combination of initial sliding of the tension rods with compression at the center.

It will be seen from these data that the Unit beams stood up as well under the load as the monolithic beam, so that the joint between slab and stem was perfectly adequate, whether corrugated or plain. The general behavior of all these beams up to the point of failure was so much the same that no one, from observation of the beams in the machine, could have pointed out which beams were unit and which monolithic. In fact, they all failed in the customary manner, exhibiting the usual inclined and vertical cracks, and no sliding was noticeable between slab and stem, although carefully looked for.

Type B, Beams 25 *and* 27. *See Figure* 163.

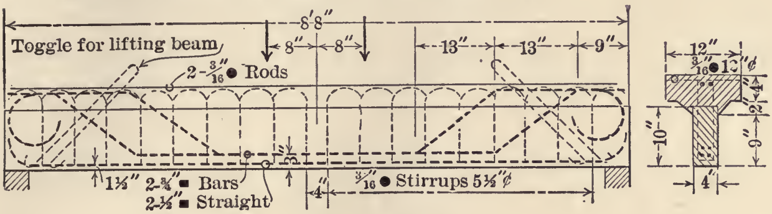


FIGURE 163.

The beams of Type B were built exactly as the beams of Type A, except that the tension rods had been reversed, being $2\frac{3}{4}''$ bars bent and $2\frac{1}{2}''$ bars straight. This reinforcement would, under the theories advanced in this book, be more efficient, and the U-bars were therefore reduced from $\frac{5}{16}''$ round stock in Type A to $\frac{3}{16}''$ round stock in Type B, thus having about one-third of the area of the former.

Beam No. 25 of Unit construction had a stem twenty-nine days old and a slab twenty-five days old; the first crack was observed under a load of 9,000 lbs., and ultimate failure took place under simultaneous compression of the slab and of the side of the stem, at the point where the tension rod was bent, under a load of 42,500 lbs. (See Figure 164.)

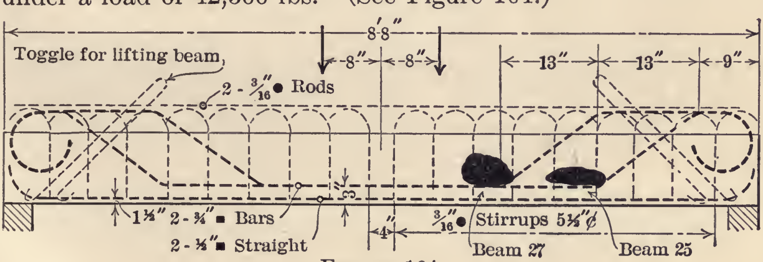


FIGURE 164.

Beam No. 27 was monolithic, of same design, and twenty-seven days old. The first crack was seen at 10,000 lbs.; while the ultimate load was 47,500 lbs. Also in this case was compression in both slab and stem evident as shown in Figure 164.

It is interesting that these two beams carried practically as much load as the older beams of Type A, in spite of the great reduction in the weight of the U-bars. The explanation is to be found in the theory set forth in Chapter VII of this book, where the relation between the bent bars and the U-bars has been considered at length; — in fact, the design of beams 25 and 27 was made to prove, or disprove, these theories as far as possible.

Type C, Beams 13 *and* 20. *See Figure* 165.

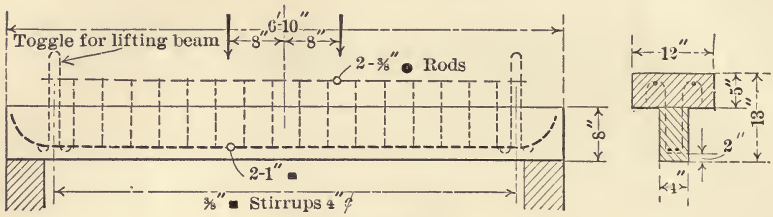


FIGURE 165.

In designing these beams, Professor Johnson had endeavored to secure a high strength in compression and tension. The special feature was the absence of bent bars so that the stresses in the stem and in the joint between slab and stem were especially severe under the common theory of shear. The beams rested in concrete supports shown in Figure 166.

Beam No. 20 was of the Unit type with corrugated top (Figure 167). The stem was forty-two days old, the slab thirty-

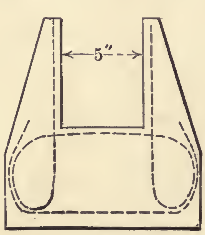


FIGURE 166.

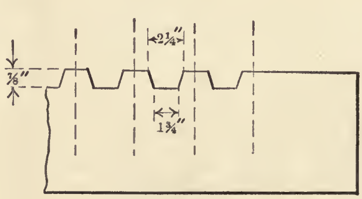


FIGURE 167.

two days old. Tension cracks developed in the usual manner, beginning under a load of 10,000 lbs.; at 22,000 lbs. small diagonal cracks appeared. At 50,000 lbs. it was noticed that the visible end of the curved tension rods began to slide, and the ultimate failure occurred under a load of 55,600 lbs., when the compression area was crushed.

Beam No. 13 was again of the Unit type, with smooth top of stem, which was forty-three days old; the slab was thirty-four days old. The first tension crack occurred at 9,000 lbs., and the cracks then developed in the usual manner. Failure took place at 50,000 lbs., when the adhesion between concrete and steel was broken; the rods began to pull through, and the slab was crushed at the center.

The analysis of the stresses follows:

Types A and B

By reference to formula (18), page 38, we have

$$\frac{n}{b} = \frac{1}{3}; \text{ T} = 4''; \text{ H} = 9''; \text{ D} = 13''; \text{ V} = \frac{15}{12} \times 1.62 = 2$$

hence

$$\frac{S}{C} = 15 \cdot \frac{82.0}{63.25} = 19.4$$

and

$$x = \frac{1}{1 + \left(\frac{19.4}{15}\right)} = \frac{1}{2.29} = .436; \; 1 - \frac{1}{3}x = .885$$

Now, the bending moment is $\frac{1}{2} \cdot L.42 = 21\ L$, and the arm of internal stresses approximately

$.855 \times 13 = 11.1$ inches, hence the pull in the steel

$$s = \frac{21}{11} \cdot L = 1.89L \text{ lbs.}$$

The beams had 1.62 square inches of tension steel, hence the unit tension on steel:

$$S = \frac{1.89}{1.62} \cdot L = 1.17 \cdot L$$

and the unit compression on the concrete

$$C = \frac{1.17}{19.4} \cdot L = .062 \cdot L$$

Type C

$$\frac{n}{b} = \frac{1}{3}; \; T = 5''; \; H = 6''; \; D = 11''; \; V = 15 \cdot \frac{2}{21} = 2.5;$$

hence

$$\frac{S}{C} = 15.9$$

and

$$x = \frac{1}{1 + \left(\frac{15.9}{15}\right)} = \frac{1}{2.06} = .485;\ 1 - \frac{1}{3}x = .838$$

Again, the bending moment is $\frac{1}{2} \cdot L \cdot 30 = 15\ L$, and the arm of internal stresses approximately

$11 \times .838 = 9.2''$, hence the pull in the steel is

$$s = \frac{15}{9.2} \cdot L = 1.63\ L$$

The beams had 2.0 square inches of steel, hence the unit tension on steel

$$S = \frac{1.63}{2.0} \cdot L = .815\ L$$

and the unit compression on the concrete

$$C = \frac{.815}{15.9} \cdot L = .0512\ L$$

It is evident that these calculations do not give the true stresses existing at rupture, because r is not equal to 15 at that time, and the assumption of plane sections probably does not hold good. For the sake of comparison, however, they may be useful. The results are indicated in the table. The testing

Type of Reinforcement	Beam No.	How made	Age (days)		Ultimate Load lbs.	Corresponding calculated stresses 165 sq. in.	
			Stem	Slab		C	S
A	1	Unit, Smooth Top	46	37	49,400	3060	57,800
A	2	Unit, Smooth Top	46	37	54,300	3370	63,500
A	4	Unit, Smooth Top	45	36	48,000	2980	56,200
A	5	Unit, Corrugated Top	44	35	41,000	2540	48,000
A	9	Monolith	41	41	50,000	3100	58,500
A	10	Monolith	41	41	52,400	3240	61,200
A	12	Monolith	41	41	45,800	2840	53,600
B	25	Unit, Smooth Top	29	25	42,500	2640	49,700
B	27	Monolith	27	27	47,500	2950	55,600
C	13	Unit, Smooth Top	43	34	50,000	2560	40,700
C	20	Unit, Corrugated Top	42	32	55,600	2840	45,300

Harvard Test Beams. Summary of Results obtained machine was equipped with means for registering the deflections automatically; the diagrams are shown in Figures 168,

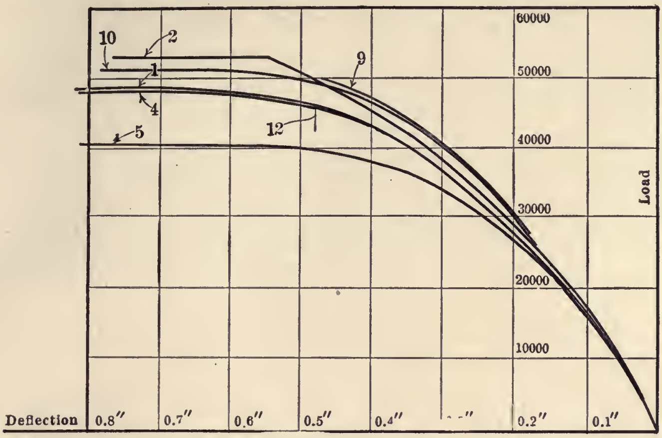


FIGURE 168.

169, and 170. Generally speaking, there is little difference in the deflection of the Unit and monolithic beams.

A number of interesting observations were made during these tests. *First*, the feasibility of the Unit beam was established beyond doubt, contrary to what many engineers would

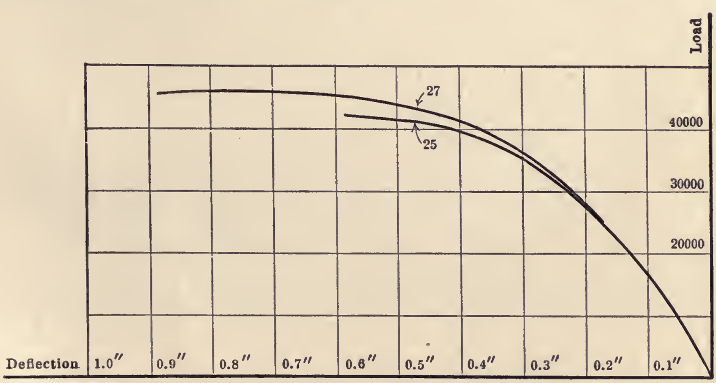


FIGURE 169.

probably have expected. In fact, many building regulations throughout the country specify positively that the beam and its superimposed slab must be concreted in one continuous operation. Where improperly designed, or otherwise inadequate, U-bars are used, this rule is undoubtedly highly beneficial, but where proper U-bars are used, the rule is wholly unnecessary. The progress report of the special committee of the American Society of Civil Engineers recommends that the slab be considered effective in compression when "proper bond" is provided between slab and stem; it will be appreciated that this is a much more consistent requirement, although somewhat indefinite. The beams tested so far have shown that the bond provided was adequate, whether the more elaborate method of

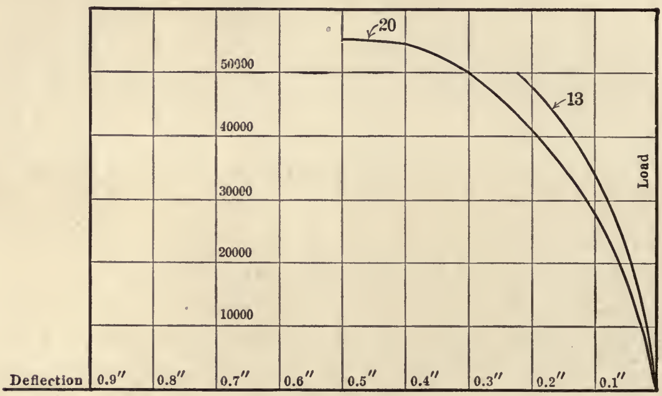


FIGURE 170.

corrugating the top of the stem was used, or whether the top of the stem was simply left as it was upon completion. It would be very interesting to learn what would happen when the bond was "inadequate," and just where the limit may be found, and in this particular the present tests furnish no information, as the bond remained intact in all cases. See Figures 171 and 172, showing the Unit Beam No. 25.

In the *second* place, these tests confirm in a remarkable degree the theories set forth by the author in Chapter VII in regard to the action of U-bars.

Compression failures of the stem were observed in beams 25 and 27; these are shown in Figure 164 and in Figures 171–174. It was observed that the compression failure of the stem was on the same side as the corresponding bent bar, the two bent bars being each near the opposite face of the beam; in beam 27

FIGURE 171. HARVARD BEAM NO. 25.

The black lines are ink marks indicating the principal cracks

Photo by Mr. J. R. Nichols, Jr., Am. Soc. C. E.

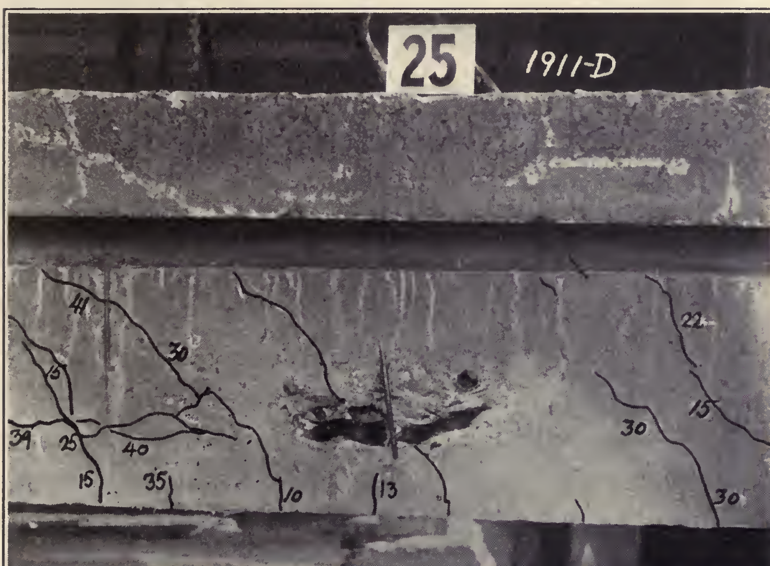


FIGURE 172. A CLOSER VIEW OF BEAM NO. 25, SHOWING CRUSHING OF THE CONCRETE AT THE ROD.

This beam was a Unit beam, — it will be noticed that there was no indication of slipping between stem and slab.

Photo by Mr. J. R. Nichols, Jr., Am. Soc. C. E.

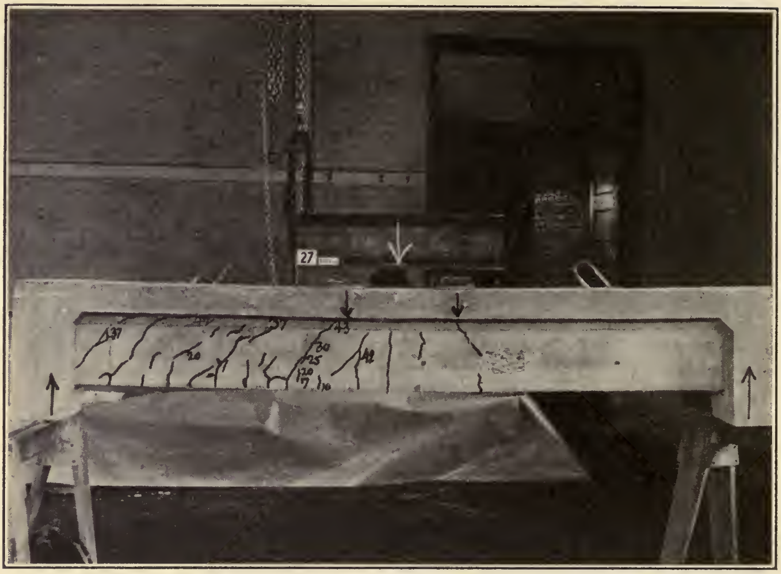


Figure 173. Beam No. 27, Showing Principal Cracks at Left End, and the Crushing of the Concrete at the Rod.

Photo by Mr. J. R. Nichols, Jr., Am. Soc. C. E.

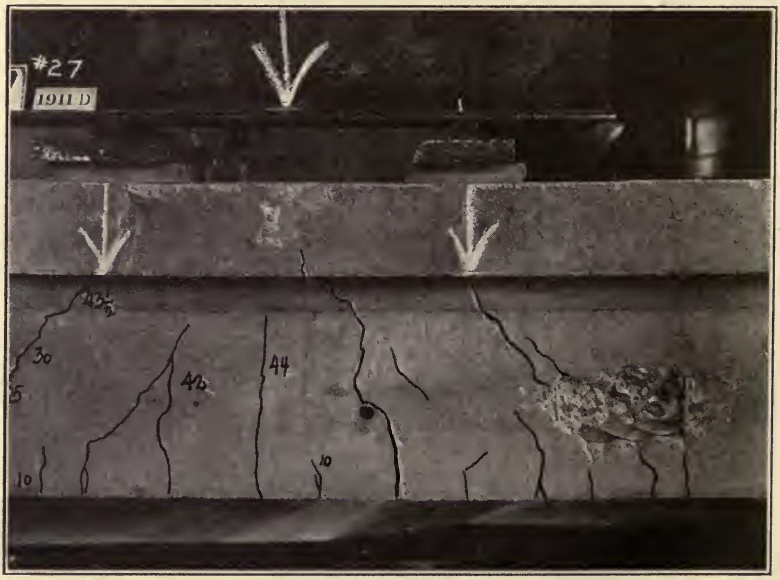


Figure 174. Closer View of Beam No. 27.

Photo by Mr. J. R. Nichols, Jr., Am. Soc. C. E.

crushing took place at both bent bars, one spot on each side, but in different locations, corresponding to the position of the curves in the bars. It is self-evident that this upward pressure of the rod must be resisted by an equal downward pressure (from the load) thus dissolving the beam into a number of well-defined compressive zones in a manner very different from what takes place in a "solid" homogeneous beam. The same observation was made by Prof. Mörsch in regard to his test beam No. VI.

Third, a deep, gaping crack was observed in the top of beam No. 20 (Figure 175), near the support. The explanation of this

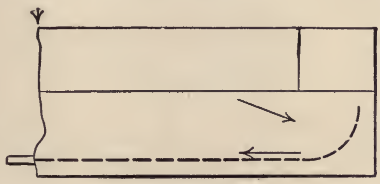


FIGURE 175.

crack may be found in the distribution of internal stresses indicated in the drawing, the horizontal arrow at the steel indicating the pulling of the steel, the inclined arrow indicating the sum of the compressive forces in the concrete. It will be noted that if these two do not intersect on the vertical line of the reaction, a "reverse" bending moment is created at the end which would cause just such a crack. Here again we have a fact showing that a reinforced concrete beam cannot be considered as a "solid" beam, in which such stresses are impossible. Considering the beam as a truss, we see at once that the crack comes outside the "end panel," and so would have no influence on the load-carrying capacity.

In addition, it must be admitted that "shear," so called, would have caused the instantaneous collapse of a beam with such a crack. As an actual matter of fact, this beam, with the gaping crack in the top, carried a total load of 55,600 lbs., *or more than any other beam of the entire series.* The stem was perforated with inclined and vertical cracks so that the only portions of the beam which could actually carry some shear were the main tension rods. This proposition has been considered above and cannot be maintained. The truth is that there was no active shear in this beam, the system consisting approximately of members as shown in Figure 176.[1]

Fourth, it was established that the quarter turn given the straight tension rods at the ends was not sufficient to develop the desired amount of sliding resistance. Thus, beams 9 and 12 failed suddenly by the entire separation of the lower rods from the concrete, while the beams of Type C (13 and 20) showed a sliding of from $\frac{1}{8}''$ to $\frac{1}{4}''$ (in these beams, the ends of the rods

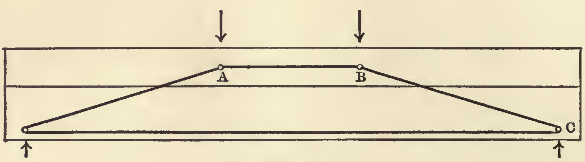


FIGURE 176.

could easily be observed by breaking away a thin shell of concrete). The behavior of the balance of the beams, and especially inspection of the deflection diagrams, makes it, however, apparent that only a very small additional margin of sliding resistance was required in order to prevent the sudden collapse. Without doubt, the large turn of the upper bar might profitably have terminated at its lowest point, as the last fourth of the circle materially weakened the concrete along the lines of cleavage

[1] The authors are aware of the fact that other observations were made during the testing of the Harvard series which strongly support the theory advanced in Chapter VII of this volume. We are, however, requested to withhold this matter from publication at the present time, and we must refer to the later report to be published by Prof. Johnson.

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