REINFORCED CONCRETE CONSTRUCTION

HENRY ADAMS

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REINFORCED CONCRETE CONSTRUCTION

IN THEORY AND PRACTICE

AN ELEMENTARY MANUAL FOR STUDENTS AND OTHERS

BY

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WITH ILLUSTRATIONS

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PREFACE.

ALTHOUGH the word theory is used in the title the Authors have intentionally avoided all pure theory, and have included only those theoretical principles which are needed to enable the necessary calculations in the practical designing of such work to be understood. The formulæ have been reduced to the simplest conditions, and worked examples are given so that the merest tyro in mathematics will have no difficulty in utilising them.

The illustrations, which are very numerous, are grouped under the various branches of construction so that anyone interested in a particular subject can study a variety of typical forms. Many of the illustrations are published for the first time, and the Authors have to thank several kind friends for permission to publish others, and for the loan of blocks.

Although there are other books dealing with both the theory and practice of reinforced concrete construction, there is none that takes it quite in the same way, and the Authors trust that the facility this offers for practical use will be found one of its chief recommendations.

CONTENTS.

CHAPTER I.

CHAPTER II.

CHAPTER III.

CHAPTER IV.

CHAPTER V.

CHAPTER VI.

CHAPTER VII.

CHAPTER VIII.

CHAPTER IX.

CHAPTER X.

CHAPTER XI.

CHAPTER I.

HISTORY OF REINFORCED CONCRETE.

ALTHOUGH this material has only been extensively used during the past few years, its first use goes back to the time of the Romans. They undoubtedly understood its principles, for we find them using reinforced concrete a hundred years B.C., in the construction of the roof of a tomb. They inserted into the concrete bronze rods crossing each other latticewise. Their concrete was, however, very poor in quality compared with our Portland cement concrete, the aggregate used by them was very coarse, and the cementing material consisted of rich lime with the addition of volcanic scoria. The Romans used reinforced concrete in many other ways, sometimes inserting tiles and timber to reinforce the concrete.

In Loudon's "Encyclopædia of Cottage, Farm and Villa Architecture" published in 1830, he suggests that it is quite feasible to strengthen concrete flat roofs by inserting a lattice-work of iron rods, while in 1840 M. Louis Leconte took out a patent for a reinforcing ceiling slab. At this time there were two systems of reinforced concrete floor construction employed in Paris, these were known as the Vaux and Thuasné systems. The reinforcement in the former case consisted of round rods hooked over wrought iron flat bars, and in the latter case of rods suspended by means of stirrups from small iron joists. Portland cement, which had only been invented in 1824, and of which very little had been manufactured, was not used in either of these floors, but plaster of Paris which was altogether unsuitable, the result being that rusting of the metal occurred, and the system was looked upon as being unsatisfactory.

It is unnecessary to enumerate the many patents that have been taken out since that time, (there being over seventy systems in use to-day, that is in Europe and America) or to refer to the scores of papers on the subject which have been read, or articles dealing with it which have appeared; it should, however, be explained that the first two real inventors of reinforced concrete were Wilkinson and Coignet. Wilkinson was a plasterer residing at Newcastle who took out a patent in 1855 for the reinforcing of concrete slabs by the insertion of a network of flat iron rods placed on edge, or as an alternative wire ropes (second hand). His object seems primarily to have been to produce a floor that would resist fire. His floors were constructed either in the arch form or flat, and from the placing of the reinforcement it is evident that he quite understood modern principles of reinforced concrete construction.

Coignet was a Frenchman—a contractor of Paris, who took out an English patent in the same year (1855) as well as patents in France. His system consisted in the reinforcing of a slab by placing iron rods crosswise, lattice fashion. He constructed twenty-eight large arches in the aqueduct of the River Vanne for the Paris water supply, reinforcing his concrete by the insertion of iron rods, but upon what method is unknown. This aqueduct is still in excellent condition. In 1861 Monier, a Parisian gardener, constructed tanks and tubs of reinforced concrete, and at the Paris Exposition of 1867 both he and Coignet exhibited specimens of their work in this material. Since then, a large number of systems have been placed on the market, including the well-known Hennebique, Kahn, Indented bar, Thacher bar, Ransome bar, Considère, Expanded Metal, and Lock-woven Mesh Systems.

Use and Advantages of the Material.—Reinforced concrete has been extensively used in America, on the Continent of Europe, and also very largely in this country; it is a material which is eminently suitable for use in engineering and architectural structures. It has been used very considerably in the construction of bridges, culverts, retaining walls, dams, reservoirs, aqueducts, conduits, sewer pipes, water mains, wharves, jetties, lighthouses, warehouses, silos, bins, and a score of other engineering works; it has also been used very largely in building construction, being most suitable for all heavier structures such as warehouses, factories, hotels, public buildings, etc. In the construction of beams, floor slabs, columns, piles, cantilevers, stairs, flat roofs, arches, etc., it is an ideal material to use.

Why is this material so serviceable? It is because its strength, rigidity, durability, lightness, ability to resist vibration, and fire-resisting properties are unsurpassed by any other material; it also incurs no cost in maintenance. Added to these advantages are two others, namely, that it is an economical material to use as regards initial cost, and that works can be executed very rapidly where it is employed.

Properties of Reinforced Concrete.—*Properties of Concrete.* (*a*) *Cement.*—Only the very best Portland cement should be used in reinforced concrete, and this should meet the requirements set out in the "1910 Amended British Standard Specification for Portland Cement" or those of the American Society for Testing Materials.

In America, natural cement has been used very considerably, but this is unsatisfactory, an artificial cement being far preferable. The percentage of lime in good Portland cement should not exceed 62, and it is of the utmost importance that the cement should be finely ground, as the finer it is, the greater the strength.

(*b*) *Storage of Cement.*—It is very desirable that cement should be stored in the original packages in a dry shed, having a wooden floor, and proper ventilation.

(*c*) *Time of Setting.*—Generally speaking a slow-setting cement should be chosen in preference to a quick-setting cement.

Broken Stone and Gravel.—The choice of the aggregate both as regards its hardness and size is a matter of the greatest importance. Broken stone (granite, trap, or other hard rock) or gravel are suitable, provided the utmost care is taken in the choice of either. Coke breeze and furnace clinker are not satisfactory, and some object to broken brick, although this is far superior to the two former. If the work is to be watertight, gravel concrete has an advantage over broken stone owing to the fact that when gravel is used a lesser proportion of voids occurs, as the rounded shape of the particles allow of a closer association, and it is specially suitable for foundation work and mass concrete. It has been estimated that gravel concrete averages $7\frac{1}{2}$ per cent less in voids than crushed stone.[1]

Limestone should not be used where the finished work may have to resist fire, as this stone suffers severely by calcination; gravel, broken sandstone, and broken brick are the best aggregates to use for work of this class.

The size of the aggregate should be variable, but such that the whole of it will pass through a $\frac{3}{4}$ in. ring. If broken stone is used it should be well screened before use so as to remove dust. If gravel is used it is desirable that it should vary in size from say $\frac{1}{4}$ in. to $\frac{3}{4}$ in.; material passing through a $\frac{1}{4}$ in. sieve should be considered as sand.

Sand.—It is preferable that this should be of fairly coarse grain; 75 per cent of it, however, should be capable of passing through a $\frac{1}{8}$ in. square mesh.[2]

It is very important that it should be absolutely clean and free from any clay, chalk, lime, or earth. Sea sand, or coarse gritty river sand are suitable. A very fine sand is undesirable.

Proportions.—The materials should be measured separately in gauge boxes. The proportions generally used vary from 1 : $1\frac{1}{2}$: 2 to 1 : 2 : 4 and 1 : 3 : 6, these representing cement, sand, and gravel or broken stone respectively. The nature of the work will largely govern the proportions of ingredients to be used. For example, a reservoir floor or bottom, and walls, should be of concrete of the proportions of 1 : $1\frac{1}{2}$: 2 or 1 : 2 : 3, while if the concrete is for the roof of a covered service reservoir 1 : 2 : 4 will do admirably.

Weight of a Cubic Foot of Cement.—For the purpose of proportioning the amount of cement to be added, this may be taken at 90 lb.

Weight of a Cubic Foot of Concrete.—145 lb. may be taken as the approximate weight of a cubic foot of concrete, and 150 lb. to 156 lb. the weight of a cubic foot of concrete including the reinforcement. 150 lb. will undoubtedly be sufficient in most cases; it will seldom occur that more than 5 lb. of re-

[1] Paper by Hewson, Architects' Business Association of Chicago.

[2] "Provisional Report on Reinforced Concrete," R.I.B.A.

inforcing steel is added per cubic foot of concrete[1] but in certain rare cases where heavy reinforcement is necessary the weight may equal 156 lb.

Mixing.—Concrete should be mixed in small batches, the proportions having been accurately measured. It should be laid as soon as mixed, and rammed or tamped vigorously till moisture appears at the surface, and all air bubbles are excluded and voids filled.

Hand Mixing.—When concrete is mixed by hand the materials should be turned over at least twice dry or until the colour is uniform, and twice wet on a wooden platform.

Machine Mixing.—This is preferable to hand mixing. There are many excellent concrete mixing machines in use to-day in this country and in America; some of these are briefly described in Chapter XI.

Laying.—The thickness of loose concrete that is to be punned should not exceed 3 in. before punning. Special care should be taken to ensure perfect contact between the concrete and the reinforcement; the punning should be continued until the concrete is thoroughly consolidated without disturbing the position of the reinforcement. Each section of concreting should as far as possible be completed in one operation; when this is impracticable, and work has to be recommenced on a recently-made surface it is necessary to wet the surface, and where it has hardened it must be hacked over, swept clean and covered with cement grout. The concrete when laid should be protected from the action of frost, and shielded against too rapid drying from exposure to the sun's rays or winds, and kept well wetted. All shaking and jarring must be avoided. The efficiency of the structure depends chiefly on the care with which the laying is done.[2]

Consistency.—No definite rule can be laid down as to the proportion of water which should be added in mixing concrete, but the concrete should always be sufficiently plastic to thoroughly fill the mould, and to fit very compactly around the reinforce-

[1] "Preliminary and Interim Report of the Committee on Reinforced Concrete, Inst. C.E.," p. 61.

[2] "Provisional Report on Reinforced Concrete," R.I.B.A.

ment. The temperature at the time of mixing will also govern the amount of water to be added.

In America what is known as a "wet mixture" is invariably used. It was thought at one time that the greater the quantity of water added, the less the strength of the concrete, but exhaustive experiments carried out in 1896 by Mr. Geo. W. Rafter, M.Am.Soc.C.E., for the State Engineer's office of New York, indicate that a "wet mixture" is not materially weaker than a "plastic mixture," whilst it fills the moulds more easily.

The average results obtained by Mr. Rafter were as follows:—

	Compressive Strength.
Dry mixture . .	2470 lb., per sq. in.
Plastic mixture .	2294 ,, ,, ,,
Wet mixture . .	2180 ,, ,, ,,

The number of blocks tested in the first case was 156; in the second 144; and in the case of the "wet mixture" 148; each block consisted of a 12 in. cube; the age of the blocks varied from eighteen to twenty-four months.

English engineers and architects do not look with favour upon a "wet mixture," but as a rule advocate a "plastic mixture," and less often a "dry mixture". The authors favour a "plastic mixture" which will quake under a moderate amount of punning, but in reservoir work, or any work which is to be watertight, a "wet mixture" may be used. In fact wetter mixtures are now more used than formerly.

In work which is to be fireproof, as in the case of tall chimneys, a "dry mixture" should be used.

Sea water should *not* be used except for mass concrete on marine works.

Crushing Strength.—In the British Standard Specification there is no compression test included; this, in the authors' opinion is an unwise omission, as concrete is so largely used for foundations, columns, etc., where it is subject to severe compression.

The recommendations of the R.I.B.A. Joint Committee are that the crushing strength of concrete of a mixture of 1 : 2 : 4 (cement, sand, and hard stone) should be not less than 2400 lb. to 3000 lb. per square inch after twenty-eight days; this appears to be reasonable, for exhaustive tests made by Mr. Geo. A. Kimball, chief engineer of the Boston Elevated Railway Company,[1] show that concrete blocks of a mixture of 1 : 2 : 4 at a period of one month after mixing crushed at an average of 2399 lb. per square inch. At three months similar blocks crushed at 2896 lb., and at six months at 3826 lb.

The Building Regulations of New York, Buffalo, San Francisco, and Cleveland specify that the safe working stress on concrete in compression shall be taken as 500 lb. per square inch. It is certainly desirable, before commencing the carrying out of concrete work of any importance, that test blocks should be made, say 4 in. cubes. They should be carefully prepared in moulds, and tested twenty-eight days after moulding, the load being slowly and uniformly applied. The average of the results should be taken as the strength of the concrete.

Tensile Strength.—The "British Standard Specification for Portland Cement"[2] specifies that the average breaking stress of briquettes made of neat Portland cement shall be as follows:—

At 7 days after gauging—not less than 400 lb. per square inch of section.

At 28 days the briquettes must show an increase on the breaking stress at 7 days after gauging of not less than:—

25 per cent when the 7 day test is above 400 lb., and not above 450 lb.

20 per cent when the 7 day test is above 450 lb., and not above 500 lb.

15 per cent when the 7 day test is above 500 lb., and not above 550 lb.

10 per cent when the 7 day test is above 550 lb., and not above 600 lb.

5 per cent when the 7 day test is above 600 lb.

The test for tensile strength (cement and sand, 3 to 1) is specified to be not less than:—

7 days' test.	28 days' test.
150 lb. per sq. in. of section.	250 lb. per sq. in. of section.

[1] "Tests of Metals," U.S.A., 1899, p. 717.

[2] "British Standard Specification for Portland Cement" (No. 12, Revised August, 1910), pp. 8 and 9.

The increase in the breaking stress from 7 to 28 days must be not less than :—

25 per cent when the 7 day test is above 200 lb. and not above 250 lb.

15 per cent when the 7 day test is above 250 lb. and not above 300 lb.

10 per cent when the 7 day test is above 300 lb. and not above 350 lb.

5 per cent when the 7 day test is above 350 lb.

The American Standard Specification for cement[1] demands a higher tensile strength as follows :—

NEAT CEMENT.

Age.	Strength.
24 hours in moist air	150-200 lb.
7 days (1 day in moist air, 6 days in water)	450-550 ,,
28 days (1 day in moist air, 27 days in water)	550-650 ,,
One part cement, three parts sand.	
7 days (1 day in moist air, 6 days in water)	150-200 ,,
28 days (1 day in moist air, 27 days in water)	200-300 ,,

The tensile strength of concrete may be roughly taken as one-tenth the compressive strength. The well-known tests of Prof. W. K. Hatt[2] would seem to substantiate this ; these tests were as follows :—

Mixture.	Age.	Compressive Strength in lb. per sq. in.	Tensile Strength in lb. per sq. in.
1 : 2 : 5 (broken stone)	28	2290	237
1 : 2 : 5 ,, ,, .	90	2413	359
1 : 5 (gravel) . .	28	2400	253
1 : 5 ,, . .	90	2804	290

[1] "Standard Specification for Cement—American Society for Testing Materials," pp. 7 and 8.

[2] "Journal of Assoc. Eng. Societies," Sept. 1900.

Shearing Strength.—A series of tests made at the University of Illinois in order to ascertain the shearing strength of concrete gave the following results:—

Mixture.	Compressive Strength in lb. per sq. in.	Shearing Strength in lb. per sq. in.	Ratio of Shearing to Compressive Strength.
1 : 2 : 4	3210	1418	·44
1 : 3 : 6	2290	1250	·57

Reinforced concrete beams rarely fail through shearing of the concrete, but when the shearing force exceeds 100 lb. per square inch of section, the beam often fails by "diagonal tension" in a curved crack which crosses the neutral axis at an angle of 45 degrees.[1] Diagonal reinforcement often doubles the resistance of a beam to shearing.

Modulus of Elasticity.—This may be taken as varying from 2,500,000 to 3,500,000 lb. per square inch for working loads for ordinary concrete; it will depend upon the mixture and age of the concrete. It has been suggested[2] that it is safer to take 2,000,000 lb. for most calculations.

Contraction and Expansion.—Considère's experiments indicate that a 3 to 1 mortar will shrink from ·05 per cent to ·15 per cent if the hardening takes place in air, and extends over a period of from two to four months, and that neat cement shrinks nearly three times as much. He also discovered that by reinforcing his mortar the shrinkage was reduced to ·01 per cent.

The coefficient of expansion of concrete is about ·000006 per 1° F. change of temperature, while that of steel is ·0000064 to ·0000068, practically the same, showing that there is no danger of unequal expansion causing a separation between the concrete and the steel.

Centering or Casing.—This should be so constructed that it will remain rigid and unyielding during the placing and punning

[1] Memorandum "E," p. 101, "Report of Committee on Reinforced Concrete, Inst. C. Engineers".

[2] "Reinforced Concrete Construction," by Turneaure & Maurer, p. 25.

of the concrete. It should also be so fixed that it will admit of easy removal, and should be well greased with soft soap or oil before the concrete is laid.

Striking of Centres.—The centres should be removed most carefully to avoid vibration; on no account should they be knocked down, falling with a crash. No definite time can be fixed for their removal unless the concrete has been laid by specialists, it may vary from ten to thirty days, and it should never be less than ten days. When the concrete has been laid by skilled men under strict supervision the centering should remain until the expiry of the following periods from the placing of the concrete in position; walls and chimneys, two days; sides of beams, three days; pillars, floor slabs, and roof slabs, seven days; underside of beams, fourteen days; arches and domes, one day for each foot of span, but not less than seven days nor more than twenty-eight; for all other portions of a building not less than fourteen days. Days in which the temperature falls below freezing-point are not to be counted in the above periods.

Properties of Reinforcing Steel.—The steel used in reinforced concrete structures should answer the following requirements:—

Its ultimate strength should not be less than 60,000 lb. per square inch.

Elastic limit 50 to 60 per cent of the ultimate strength.

Modulus of elasticity, 30,000,000 lb. per square inch.

Working stress, factor of 4 } according to the nature of the
,, ,, ,, 5 } structure.

The steel should afford an elongation of not less than 22 per cent in round bars less than one inch in diameter on a gauge-length of 8 diameters, and should stand bending cold 180 degrees to a diameter of the thickness of the pieces tested without fracture on the outside of the bent portion. All rods, plates, bars, and other reinforcement should be of mild steel, manufactured on the open hearth basic or acid Siemens process.

Plain and Deformed Bars.—In Europe plain bars have been chiefly used, but in America deformed bars are looked upon with much favour, it being contended that the deformation in the bar tends to increase the mechanical bond. With this in view many devices have been introduced, such as twisting square rods, making round bars of alternate round and flat sections but with the same sectional area at every point. Some engineers have preferred I joists of light section, others flat bars.

In the illustrations here given :—

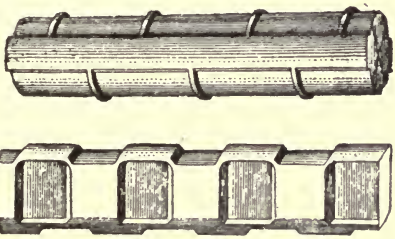


Fig. 1.

Fig. 2.

KAHN TRUSSED BAR.

Fig. 3.

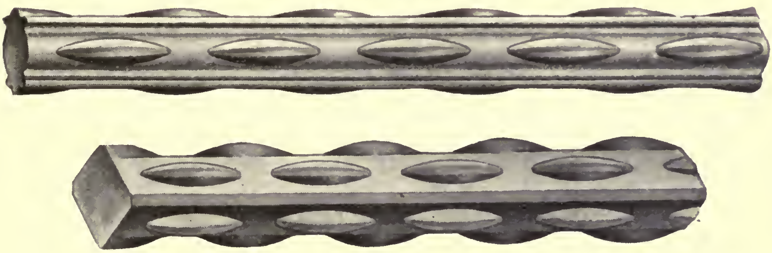


Fig. 4.

Fig. 1. Represents the Indented Steel Bar Co.'s patent bars.

Fig. 2. Ransome bar (both of these have been largely used in America).

Fig. 3. Kahn bar (Trussed Concrete Steel Co., Ltd.). This bar is formed by shearing and turning up the side wings of the bar.

Fig. 4. Havemeyer bars.

Fig. 5. Cumming's bar.

Fig. 6. Lug bars.

Fig. 6A. Another variation in twisted bars.

Other methods of reinforcing concrete consist of the insertion of Figs. 7 and 8, expanded metal; Fig. 9, lock-woven wire fabric; Fig. 10, Messrs. Wm. Moss and Sons' patent. The Thacher bar, invented by Mr. Edwin Thacher, M.Am.Soc.C.E. has been largely used in America.

Relation Between Concrete and Steel.—*Adhesion of Concrete to Steel.*—A great many tests have been made in order to determine the adhesion of the concrete to the reinforcement. Among the most recent of such tests were those carried out a short time ago at the Gewerbe Museum, Vienna. The test blocks consisted of 8 in. cubes, and the reinforcing steel of $\frac{3}{4}$ in. steel bars (round). In order that the tests should be varied, four different mixtures of concrete were used, namely: 1 : 4; 1 : 6; 1 : 8; 1 : 12, and the bars were drawn out after a period of six weeks from the time of moulding. The mean adhesive strength for the four mixtures was as follows:—

Proportions.	Adhesive strength.
1 : 4	656 lb. per square inch.
1 : 6	646 ,, ,, ,, ,,
1 : 8	601 ,, ,, ,, ,,
1 : 12	448 ,, ,, ,, ,,

The ultimate compressive strength of the concrete at a period of seven months from time of moulding was:—

Proportions.	Adhesive strength.
1 : 4	2176 lb. per square inch.
1 : 6	1834 ,, ,, ,, ,,
1 : 8	1450 ,, ,, ,, ,,
1 : 12	1052 ,, ,, ,, ,,

These results indicate that ordinary plain bars are quite satisfactory for reinforced concrete work. It will be noted from the result of the above experiments that the adhesive strength depends largely on the quality of the concrete. Other interesting tests include those carried out by Mr. Withey in 1907 at

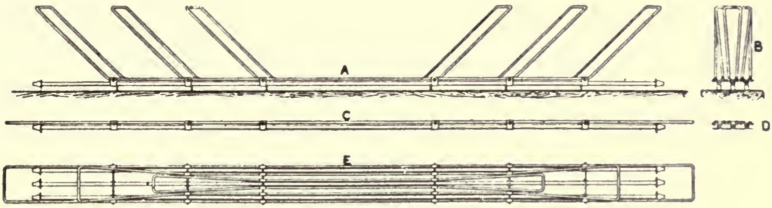


Fig. 5.

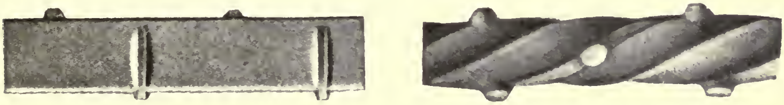


Fig. 6.

Fig. 6a.

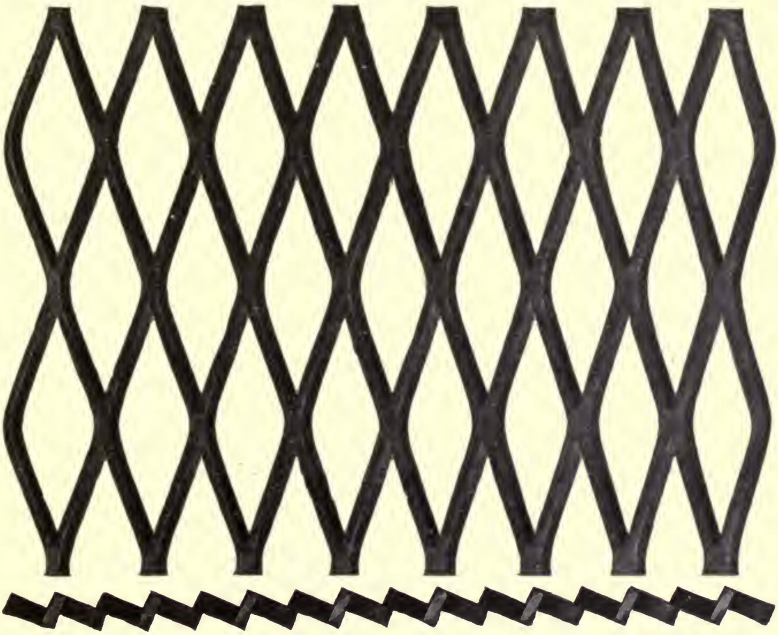


Fig. 7.

the University of Wisconsin.[1] The concrete mixture in this case was 1 : 2 : 4; plain steel rods were used varying from $\frac{3}{16}$ to

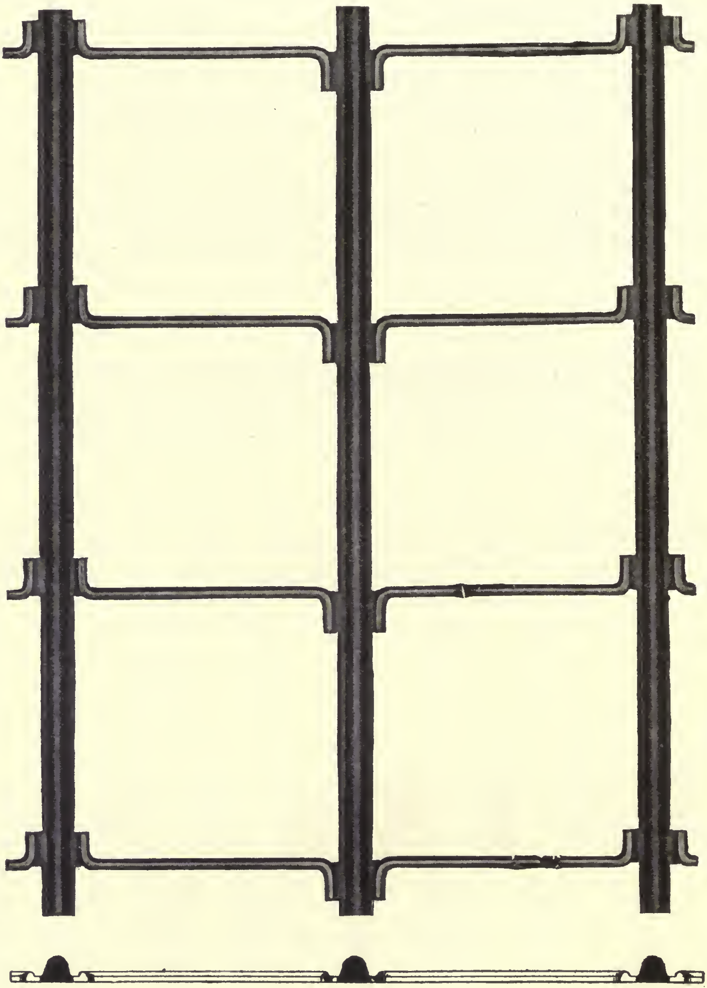


Fig. 8.

$\frac{3}{4}$ in. in diameter. The rods were embedded to a depth of 6 and 8 in. respectively, and the average adhesive strength was

[1] "Bull. Univ. of Wisconsin," 1907.

found to be 400 and 310 lb. per square inch respectively. These results are not so high as those obtained at Vienna.

A reasonable objection has been taken to all these tests on the ground that when the steel is put under tension the pro-

FIG. 9.

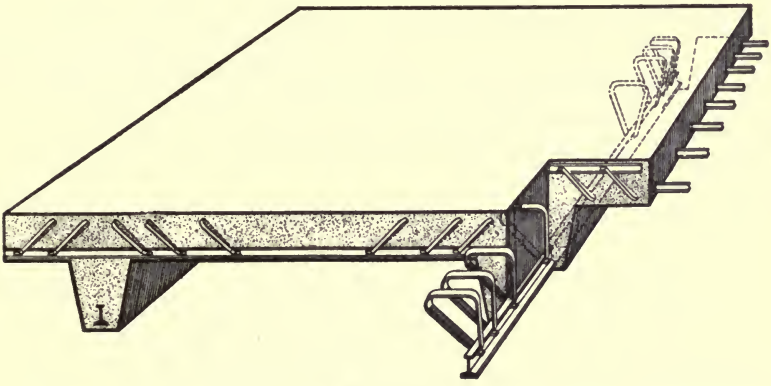


FIG. 10.

jecting part is elongated by the stress and that a progressive separation is caused between the steel and the concrete, but it is difficult to devise any method of testing that would be entirely free from objection. It would be safe to assume that with ordinary round rods, not too smooth, embedded in good concrete, the bond strength may be taken at 250 to 300 lb. per square inch.

The safe working adhesion is usually taken at 60 to 100 lb. per square inch, and it is always advisable that the ends of the rods should be split or bent as a precaution against sliding.

The working stresses allowed by the regulations of various Governments are as follows:—

Austria	1 : 3 concrete	78 lb. per square inch.
,,	1 : 4 ,,	78 ,, ,, ,, ,,
,,	1 : 5 ,,	64 ,, ,, ,, ,,
Germany	. . . .	64 ,, ,, ,, ,,
United States	. . .	50 ,, ,, ,, ,,

Position of Reinforcement.—On no account should the reinforcement be placed nearer the face of the concrete than ½ in. in slabs, 1 in. in cross beams, and 1½ in. in main beams and pillars, and not less than 2 in. in structures exposed to the action of sea water.

Preservation of Steel in Concrete.—There are still a few engineers and architects who look with disfavour upon reinforced concrete on the grounds that corrosion of the steel is likely to take place, and that being encased in the concrete that corrosion cannot be detected as it might be in an open steel structure. The authors would emphatically state that, in their opinion, where good concrete surrounds the reinforcing steel, no such fears need be entertained. Many experiments have been carried out in connection with this important matter, and what is even a more reliable test of fitness, the condition of steel wherever found to be embedded in concrete in buildings or other structures which have been demolished, has been carefully noted.

One of the authors, for example, had occasion recently (1910) to pull down a partially underground public convenience at Bridlington, which was erected fifteen years ago; the roof was reinforced by steel joists and iron rods, and these were found to be in excellent condition, there being no sign of corrosion, except where the end of one of the joists projected 4 in. beyond the face of the concrete; this exposed part was greatly corroded, but the corrosion ended flush with the face of the concrete. He also had occasion two years ago (1908) to strip the face off a concrete-in-situ sea wall at Bridlington. This wall had been erected twenty-five years previously, and was reinforced by iron chains. These chains came as near as 1 in. in places to the surface of the sea wall, and, in spite of the fact that the lower half of the wall was covered by the sea at each tide, the chains were quite bright, and no signs of corrosion were detected.

In August, 1907, a concrete pier was constructed at some works at Newark, N.J.; part of this was removed in October, 1910, and the reinforcement was found to be perfectly clean and bright, regardless of the fact that the concrete was practically under water. The mixture was a sloppy or wet one, and of the following proportions, one of cement, two of sand, and four of $\frac{3}{4}$ in. stone.

Prof. C. L. Norton of the Massachusetts Institute of Technology has carried out some exhaustive experiments in connection with this matter, and his conclusions are as follows:—

1. "That neat Portland Cement, even in thin layers, is an effective preventative of rusting.

2. Concrete, to be effective in preventing rusting, must be dense and without voids and cracks. It should be mixed quite wet when applied to the metal.

3. The corrosion found in cinder concrete is mainly due to the iron oxide, or rust, in the cinders, and not to the sulphur.

4. It is of the utmost importance that the steel be clean when embedded in concrete. Scraping, pickling, a sand-blast, and lime should be used, if necessary, to have the metal clean when built into the wall."

Note.—The authors do *not* recommend the use of breeze or cinder concrete.

The tests of German engineers[1] might be referred to as being particularly interesting, for they show that cracks occurring in the concrete do not result in the reinforcement becoming corroded at these points, and that only those beams that were

[1] Paper on "The Corrosion of Steel Reinforcement in Concrete," by E. R. Matthews. Trans. Soc. of Engineers.

stressed to more than 35,000 lb. to the square inch showed any corrosion.

The method of testing was briefly as follows :—

Beams were prepared which were reinforced by the steel which is generally used in reinforced concrete structures, this steel being placed $1\frac{1}{4}$ in. above the bottom of the beams. The concrete was mixed in the following proportions: 1 : 2 : 4, 10·1 per cent of water being added. The forms were removed after a set of twenty-four hours; the beams were then stored for three months, some in wet sand, others in the open air, water being poured over them daily. For the next three months the beams were subjected to load tests, and of fifty-eight beams made, twenty-six were broken under these tests. The thirty-two beams which had not been broken were then subjected to a rusting test under a load, sheet-iron casing being fixed around the middle third of the beam, through which a mixture of carbon dioxide, oxygen and water vapour passed. The beams were kept in this rusting atmosphere for three days from 7 a.m. to 4 p.m. The steel was then examined with the following results :—

In twenty-seven out of the thirty-two cases no rusting had occurred. These twenty-seven beams had been subjected to a load causing stresses of 18,000 lb. to 35,000 lb. per square inch in the steel, and the five remaining beams had been subjected to a greater stress, viz.: 35,000 lb. to 44,000 lb. per square inch.

Other experiments might be referred to as follows :—

In 1908 Sir H. Tanner had three blocks of concrete with steel inside put in the lake at St. James' Park, and left there for about a year; when one was broken the steel was found to be quite unaffected.

In 1884 when the Eddystone Lighthouse (originally constructed in 1757) was pulled down, a bundle of rods which had been accidentally left in the concrete in the centre of the lighthouse was found to be in perfect condition.

In views of the tests and observations before-named, we may safely conclude that *steel does not corrode in good Portland cement concrete,* where the load applied does not exceed the elastic limit of the material.

A thin surface coating of rust on the reinforcing bars has proved to be no detriment when the concrete is so made as to be in perfect contact with the bars. Paint or oil should under no circumstances be permitted on the bars, but they may be coated with cement grout immediately before depositing the concrete.

CHAPTER II.

GENERAL PRINCIPLES OF STRESS.

THE principal stresses in structures are those of compression, tension, and shear. The first two are so generally known that no explanation of their nature is needed, but with regard to shear, although it may be known to be a tendency to cut the material transversely, its mode of action and amount are not as commonly understood and will involve a detailed description in the proper place.

Reinforced concrete is cement concrete strengthened by steel rods in such a manner that the compressive stress will be taken by the concrete and the tensile stress by the steel, with some slight modifications of the rule to suit special circumstances which will be dealt with hereafter. The shear stress is taken partly by the concrete and partly by the steel.

The stresses above named are produced in the material by the weight of the material itself, by the action of external loads, and by the force of the wind. They are common to all structures of whatever material they may be formed, and may therefore be studied in the abstract before considering their special applications. Direct compression is concerned with a total force P and a total sectional area A to resist it, the intensity of the compression being given by $\frac{P}{A}=p$ lb. per square inch, or tons per square foot, as the case may be. Where the length exceeds about six times the least breadth this intensity of pressure will be modified by the tendency to bend, which will increase it on the hollow side of the bend and reduce it on the rounding side. Direct tension follows the same law as regards intensity $\frac{P}{A}=p$, but when the weight of a horizontal piece causes any tendency to sag, the tension will be increased on the rounding side and reduced on the hollow side.

A homogeneous rectangular beam of any given material, loaded on top and supported at the ends, is put in a state of stress as indicated by Fig. 11, the beam being drawn abnormally deep in proportion to the span to exaggerate the peculiarities. The forces there shown may, by the rules of elementary mechanics, be resolved into others acting vertically and horizontally. The compression acting on the upper half and tension on the lower half produce a shortening and lengthening respectively which may be indicated by drawing parallel transverse lines on the side of the unloaded beam and showing how these would vary in distance when the beam is bent as in Fig. 12. The variation from the original distance between the lines indicates precisely what is happening; the compression, greatest at top, reduces gradually down to zero at the centre of the depth, and the tension, greatest at the bottom, also reduces to zero at the centre of the depth where the spacing is unaltered. This line of no stress is called the neutral layer, or in cross-sections the neutral axis. That the beam is not only undergoing these simple stresses may be understood by considering the result of bending if the beam were made up of independent layers like boards. It is evident that there would be a slipping between the surfaces in contact as shown in Fig. 13, and this tendency exists in the solid beam although it cannot actually be seen. This endeavour to slip is known as horizontal shear. If again the beam were composed of blocks slightly attached to each other to make up the length, a vertical slipping would take place as shown in Fig. 14. This is known as vertical shear, and the two shears being compounded with the direct tension and compression make up the stresses shown by the curves of thrust in Fig. 11. Besides varying throughout the depth of a beam, the tension and compression vary also throughout the length, and, as these stresses are set up by bending, the measure of their value at any point of the length is known as a bending moment and the whole series is usually plotted as a bending moment diagram. A bending moment is like all other moments in being the product of a force into a leverage, and in the case of a beam, whatever cross-section be considered, the forces whether loads or reactions, tend to turn the end of the beam clockwise or anti-clockwise round the given section. The bending moment

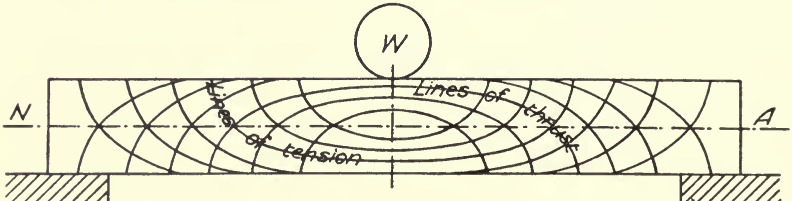


FIG. 11

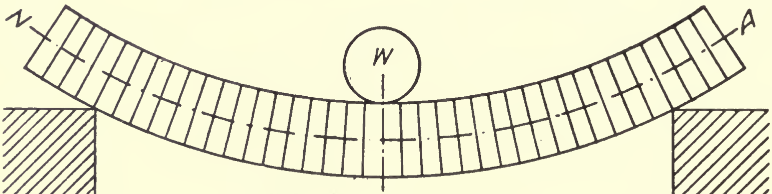


FIG. 12

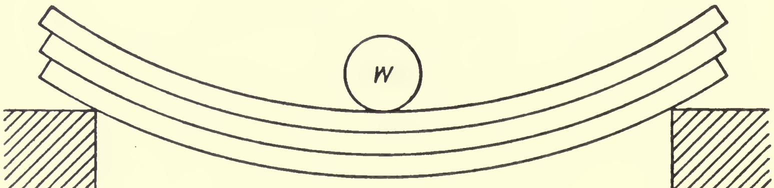


FIG. 13

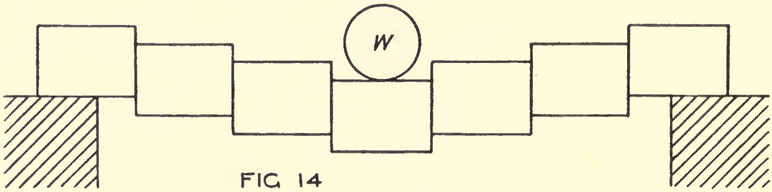


FIG. 14

FIG. 11.—Lines of stress in a rectangular beam. FIG. 12.—Lines showing alteration in length of beam under stress. FIG. 13.—Diagram showing effect of horizontal shear. FIG. 14.—Diagram showing effect of vertical shear.

at the section is the algebraic sum of the clockwise and anti-clockwise moments and is given in lb.-in., or other unit compounded of the units of the force and leverage. It is only necessary to divide the bending moment at any section by the effective depth of the beam to obtain the total stress of tension or compression at that section, and that again divided by the effective area above or below the neutral axis gives the intensity of the stress.

By Newton's third law of motion "Action and reaction are equal," and a bending moment can only exist when there is a moment of resistance to balance it.

Bending moment diagrams may be constructed to suit any condition of loading or supporting, but moments of resistance are dependent upon the material used, its strength, the sectional area and the way it is disposed round the neutral axis, and can only be determined by the exact conditions of the piece. To put it another way, the bending moment may be said to be the theoretical effort and the moment of resistance the practical opposition.

A simple cantilever loaded at the free end and built firmly into a wall at the other end may be taken for the purpose of showing how bending moments are calculated. Fig. 15 shows such a beam with a concentrated load W at the outer end, and a clear span or length l. The bending moment at any point x will be W $(l - x)$, that is, the load multiplied by its leverage to the given point. A series of such points may be taken and the results drawn to any convenient scale, and finally the whole length giving the maximum bending moment as Wl at the face of the wall. All the intermediate points being joined up will give the outline of a triangle, therefore, working graphically there will be no need to do more than calculate the maximum moment and draw in the full outline as in Fig. 16. The vertical shear at any point depends upon, and is equal to, the amount of load passing through that point to the support. In this case the whole load passes equally through each point, and the shear diagram will be shown by a rectangle with a depth $=$ W, as in Fig. 17. The formulæ for determining the bending moments and shear stresses and also the deflection have been added to the diagrams to make them more complete. In the deflection formulæ $\Delta =$ the maximum deflection in inches, W $=$ the total load in lb., $l =$ span in inches, E $=$

modulus of elasticity of the material in lb., I = the moment of inertia of the section in inch units, sometimes called the "second moment".

When more than one concentrated load is carried, as in Fig. 18, the same method will be adopted to find the bending moment and shear diagrams. Take first the outer load W, then Wl will give the maximum bending moment, the triangle occupying the

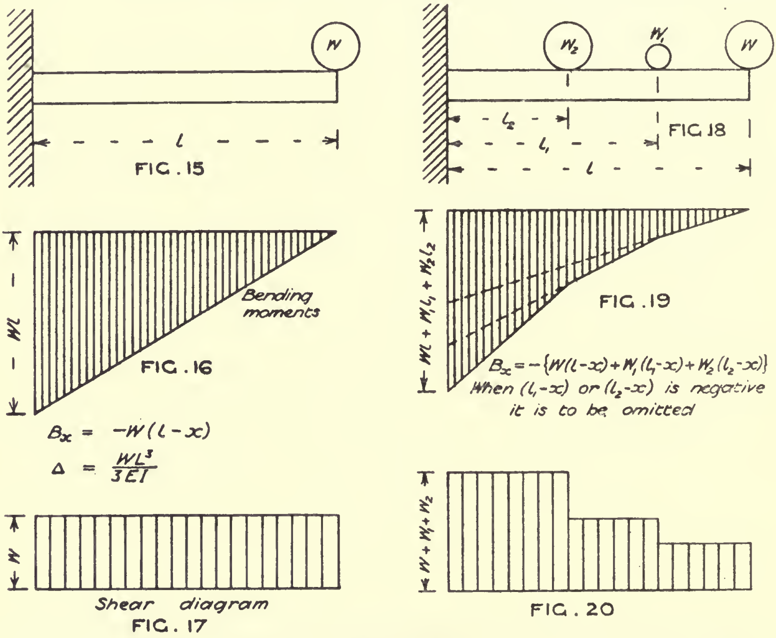


Fig. 15.—Cantilever loaded at the end. Fig. 16.—Bending moment diagram for cantilever loaded at the end. Fig. 17.—Shear diagram for cantilever loaded at the end. Fig. 18.—Cantilever with three concentrated loads. Fig. 19.—Bending moment diagram for cantilever with three concentrated loads. Fig. 20.—Shear diagram for cantilever with three concentrated loads.

whole length of cantilever as in Fig. 19. Next take the load W_1 with a leverage l_1 and add the maximum bending moment $W_1 l_1$ at the face of wall below the previous one, then complete the triangle up to the point where the load W_1 is applied. Proceed in the same way with load W_2 having the leverage l_2, and the full bending moment at any point will then be obtained by measuring the depth of the diagram at that point. It is best to commence the shear diagram at the support and then to reduce its depth as each load is passed, leaving a depth equal to the amount of load passing through the point, and giving the whole diagram as in Fig. 20.

When the load is uniformly distributed and continuous over the span, as in Fig. 21, the same principles may be adopted, and the points found for the bending moments will give the out-

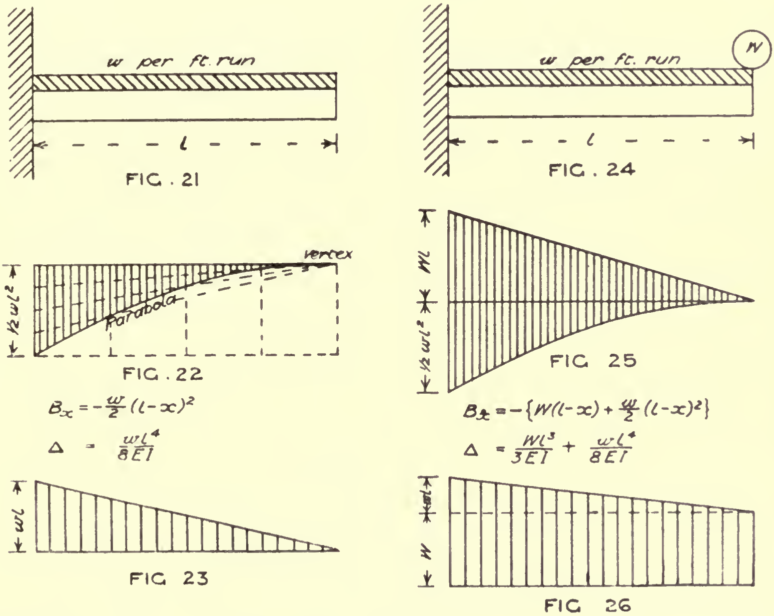


Fig. 21.—Cantilever with uniformly distributed load. Fig. 22.—Bending moment diagram for cantilever with uniformly distributed load. Fig. 23.—Shear diagram for cantilever with uniformly distributed load. Fig. 24.—Cantilever with uniformly distributed load, and concentrated load at end. Fig. 25.—Bending moment diagram for cantilever with uniformly distributed load and concentrated load at end. Fig. 26.—Shear diagram for cantilever with distributed load and concentrated load at end.

line of a semi-parabola with the vertex at the outer end, as in Fig. 22. This fact being known, the parabola may be set off with only the one calculation for maximum bending moment. A simple method for drawing a parabola by the intersection of lines is shown by dotted lines upon the same diagram. Divide the base line into any number of equal parts and draw vertical lines. Divide the height into the same number of equal parts and from each draw lines to the vertex, the intersections give points in the curve required. For the shear diagram, the same rule of starting at the support with the whole load carried there and diminishing the depth by the amount of load passed, will give the outline of a triangle as in Fig. 23.

It will be useful to collect together all the usual modes of loading and supporting and indicate the required calculations by the algebraic method of letters so that they can be readily applied to any given case.

With a distributed load and also a concentrated load upon a cantilever as in Fig. 24, the diagrams from each may be combined as in Figs. 25 and 26.

With a distributed load over a portion of the length only, as Fig. 27, the bending moment and shear diagrams will be as Figs. 28 and 29.

In a beam supported at the ends and carrying a concentrated load in the centre as Fig. 30, the bending moment diagram will be as Fig. 31 and the shear diagram as Fig. 32. The question is often asked in the latter case "what is the shear stress at the centre of the beam, from the appearance of the diagram it might be nil or double". The answer is that it is uniform throughout, the change at the centre is due to the subtraction of the whole load as it is passed, and the measurement of depth changes from above the datum line to below. Sometimes the two portions are marked plus and minus, then $+\frac{1}{2}$ W on the left $-$ W at the centre $= -\frac{1}{2}$ W on the right. In the bending moment diagram Fig. 21, the maximum occurs immediately under the load and is arrived at thus: reaction at support $=\frac{1}{2}$W, leverage to centre $=\frac{1}{2}l$, therefore bending moment

$$=\tfrac{1}{2}\mathrm{W}\times\tfrac{1}{2}l=\frac{\mathrm{W}l}{4}.$$

When the load is out of the centre, dividing the span l into the portions a and b, as in Fig. 33, the reactions at the supports must first be calculated. $\mathrm{W}\times\dfrac{b}{a+b}$ will give the reaction at A and therefore the shear at that point, while $\mathrm{W}\times\dfrac{a}{a+b}$ will give the reaction and shear at B. The maximum bending moment will be immediately under the load $= W \times \frac{a \times b}{a+b}$ as in Fig. 34, and the shear diagram will be as Fig. 35.

With two concentrated loads as in Fig. 36 there are two alternative methods of drawing the bending moment diagram.

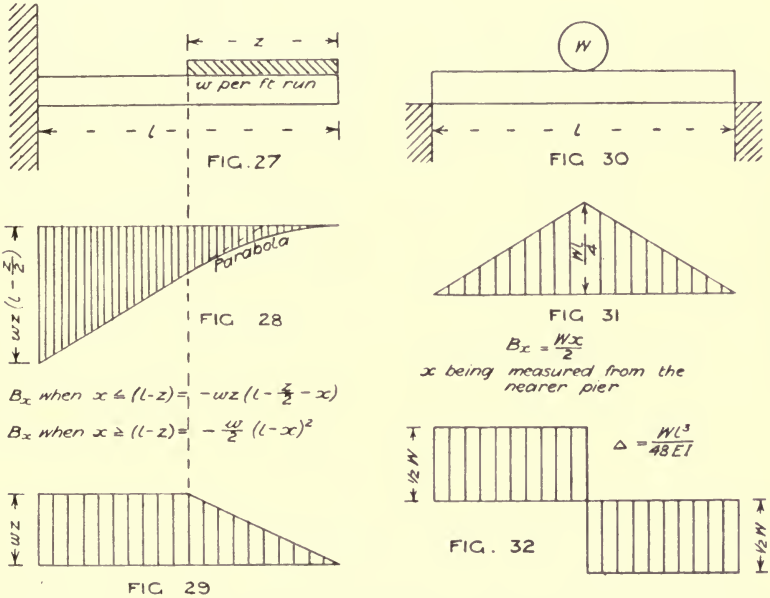


FIG. 27.—Cantilever with load uniformly distributed over a portion of its length. FIG. 28.—Bending moment diagram for cantilever with load uniformly distributed over a portion of its length. FIG. 29.—Shear diagram for a cantilever with load uniformly distributed over a portion of its length. FIG. 30.—Beam supported at the ends with load concentrated in centre. FIG. 31.—Bending moment diagram for beam supported at the ends with load concentrated in centre. FIG. 32.—Shear diagram for beam supported at the ends with load concentrated in centre.

Fig. 37 shows the two triangles set out on opposite sides of the datum line, the measurements for the bending moment at any point being taken right through. Or the triangles may be set out on the same side of the datum line as in Fig. 38, and the overlapping parts added on to the outline.

By calculation the reactions will be, at $A = \frac{W_1(b+c)}{l} + \frac{Wc}{l}$, and at $B = \frac{W(a+b)}{l} + \frac{W_1 a}{l}$. The bending moments will then be, under $W_1 = a\left\{\frac{W_1(b+c)}{l} + \frac{Wc}{l}\right\}$ and under $W = c\left\{\frac{W(a+b)}{l} + \frac{W_1 a}{l}\right\}$ The shear diagram will be constructed as before, the result being shown in Fig. 39.

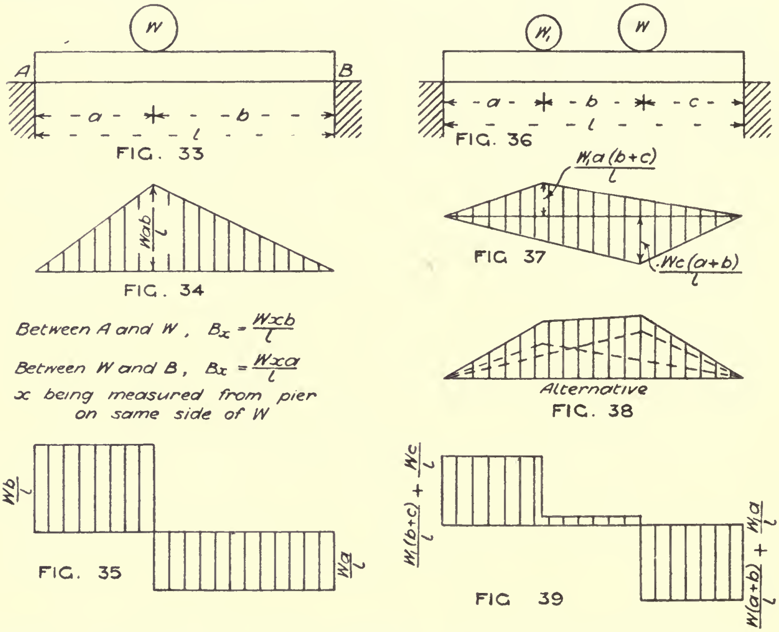


FIG. 33.—Beam with concentrated load out of centre. FIG. 34.—Bending moment diagram for beam with concentrated load out of centre. FIG. 35.—Shear diagram for beam with concentrated load out of centre. FIG. 36.—Beam with two concentrated loads. FIG. 37.—Bending moment diagram for beam with two concentrated loads. FIG. 38.—Alternative bending moment diagram for beam with two concentrated loads. FIG. 39.—Shear diagram for beam with two concentrated loads.

With two equal concentrated loads symmetrically placed on each side of the centre of span, it will be found that the bending moment will be uniform between the loads and the shear stress will be reduced to zero.

With more than two concentrated loads on a beam the graphic method of finding the bending moments will be as shown in Fig. 38, but they may also be calculated. The reaction at A multiplied by the distance to the first load, will give the bending moment under the load, and the reaction at B multiplied by the distance from it to the third load will give the bending moment under this load. The bending moment under the second load will be found by multiplying the reaction at A by the distance to the second load and subtracting the product of the first load into its distance from the second load, that is, obtaining the algebraic sum of the moments clockwise and anti-clockwise. The same procedure will be followed with any additional loads.

When the load is continuous as in Fig. 40 the calculated bending moment will be $\frac{1}{2}wl$ for the reaction, multiplied by $\frac{1}{2}l$ for the leverage to centre, minus $\frac{1}{2}wl$ for load on one half, multiplied by the distance of its centre of gravity from the centre, for the opposing moment; or

$$\tfrac{1}{2}wl \times \tfrac{1}{2}l - \tfrac{1}{2}wl \times \tfrac{1}{4}l = \tfrac{1}{4}wl^2 - \tfrac{1}{8}wl^2 = \frac{wl^2}{8}.$$

The outline of the bending moment diagram will be a parabola as in Fig. 41, and the shear diagram will be a double triangle as in Fig. 42.

It should be noted, in this and other cases of loaded beams, that the maximum bending moment and minimum shear occur at the same point, and that the bending moment at any section is equal to the area of the shear diagram up to that point measured on the length-load scale adopted, due attention being given to the plus and minus values of the shear diagram. The formula $B_{max.}$ gives the maximum bending moment and B_x the bending moment at any point x measured from the left-hand support.

With a uniformly distributed load over the whole span and a concentrated load in the centre, as Fig. 43, the bending moment diagram will be as Fig. 44, and the shear diagram as Fig. 45.

With a load distributed uniformly from one end over a portion of the span as in Fig. 46, the bending moment diagram will be as Fig. 47. Here it will be observed that there is first drawn a triangle, as if the load were all concentrated at its centre of gravity, and then, cutting off from the triangle a piece whose extremities correspond with the position of the load, upon the line so drawn across the triangle a parabola is raised with a central depth equal to what it would be if the distance z were the whole length of a girder under a uniformly distributed load, viz. $\frac{wz^2}{8}$. The shear diagram, Fig. 48, is constructed as previously described, and it will be seen that the minimum shear occurs immediately under the maximum bending moment and indicates the position where the measurement of the latter should be taken.

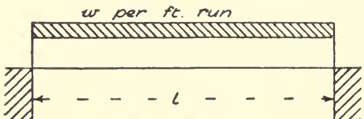


FIG. 40

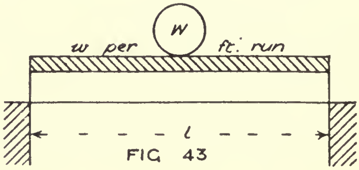


FIG 43

FIG. 41

$B_x = \frac{w}{2}(l - x)$

$\Delta = \frac{5}{8} \cdot \frac{wl^4}{48EI}$

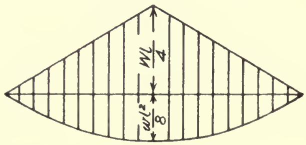


FIG. 44

$B_x = \frac{Wx}{2} + \frac{w}{2}(l - x)$

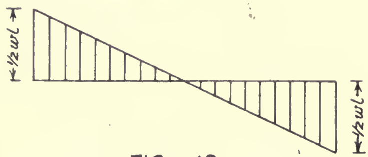


FIG. 42

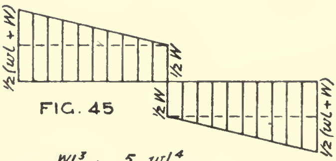


FIG. 45

$\Delta = \frac{Wl^3}{48EI} + \frac{5}{8} \cdot \frac{wl^4}{48EI}$

FIG. 40.—Beam supported at the ends with a uniformly distributed load. FIG. 41.—Bending moment diagram for beam supported at the ends with a uniformly distributed load. FIG. 42.—Shear diagram for beam supported at the ends with a uniformly distributed load. FIG. 43.—Beam supported at the ends with a uniformly distributed load and load concentrated in centre. FIG. 44.—Bending moment diagram for beam supported at the ends with a uniformly distributed load and load concentrated in centre. FIG. 45.—Shear diagram for beam supported at the ends with uniformly distributed load and load concentrated in centre.

Fig. 49 shows a girder with a concentrated load and a partially distributed load as before. The two loads shown are equal and their centres of gravity equidistant from the supports. The bending moment diagram, Fig. 50, is a combination of previous diagrams, and the shear diagram, Fig. 51, is set out as before. If the loads have been given exactly as in Fig. 49 the stress diagrams will be as shown, but if loads of other magnitudes or position are given, although of the same character, there will be a corresponding alteration produced in the diagrams, the shear diagram being more especially affected in appearance.

With a load uniformly distributed over a part of the span, away from the ends, as in Fig. 52, the bending moment and shear diagrams will be as shown in Figs. 53 and 54.

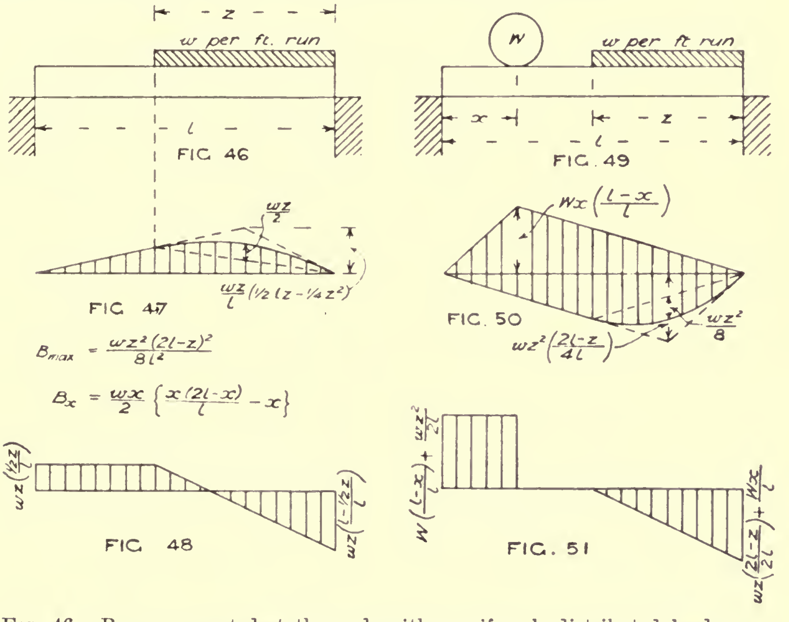


Fig. 46.—Beam supported at the ends with a uniformly distributed load over a portion of its length. Fig. 47.—Bending moment diagram for beam supported at the ends with a uniformly distributed load over a portion of its length. Fig. 48.—Shear diagram for a beam supported at the ends with a uniformly distributed load over a portion of its length. Fig. 49.—Beam with concentrated load and partial distributed load. Fig. 50.—Bending moment diagram for beam with concentrated load and partial distributed load. Fig. 51.—Shear diagram for beam with concentrated load and partial distributed load.

Beams with rigidly fixed ends, or beams continued over one or more intermediate supports, are known as continuous beams, and the stresses are much more difficult to determine than in the cases previously considered.

Fig. 55 shows a beam built into a wall at each end and loaded in the centre. Generally speaking a length of not less than four times the depth must be built in, or it must be rendered sufficiently rigid by other means, in order that the full strength may be obtained. The load will cause a concave curvature on the top of the beam in the central portion for half the span the same as occurs throughout in a beam simply supported at the ends, but the building-in of the wall ends causes a convex curvature for one-quarter of the span at each

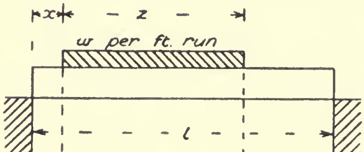


FIG 52

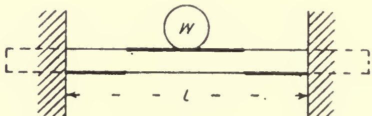


FIG. 55

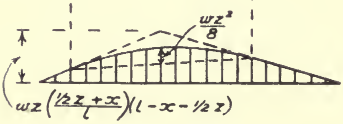


FIG. 53

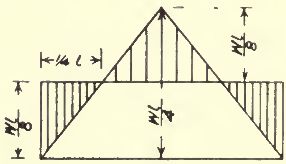


FIG. 56

$\Delta = \frac{1}{4}\cdot\frac{Wl^3}{48EI}$

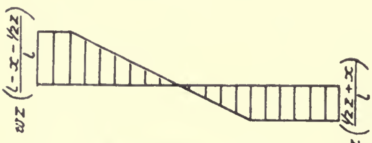


FIG 54

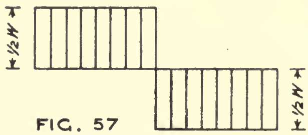


FIG. 57

FIG. 52.—Beam supported at the ends with load distributed over a portion of its length. FIG. 53.—Bending moment diagram for beam with load distributed over a portion of its length. FIG. 54.—Shear diagram for beam with load distributed over a portion of its length. FIG. 55.—Fixed beam with load concentrated at centre. FIG. 56.—Bending moment diagram for fixed beam with load concentrated at centre. FIG. 57.—Shear diagram for fixed beam with load concentrated at centre.

end, the junctions being called the points of contra-flexure. This indicates that the upper part of the beam will be in compression for the length of the concave portion, and in tension throughout the convex portions, the stresses in the lower half of the beam being reversed. The compression is shown in Fig. 55 by thickened lines. The bending moment diagram, Fig. 56, is produced by first forming a triangle as if the beam were simply supported at the ends, and then cutting off the top by a rectangle equal in height to the maximum bending moment at the supports. The positive bending moments are shown by the wide spacing of the ordinates and the negative bending moments by the narrow spacing. The shear diagram, Fig. 57, is the same as for an ordinary beam.

In the case of a continuous beam with a uniformly distributed load, as Fig. 58, there are two bending moment values, according to whether the beam is of uniform section or of uni-

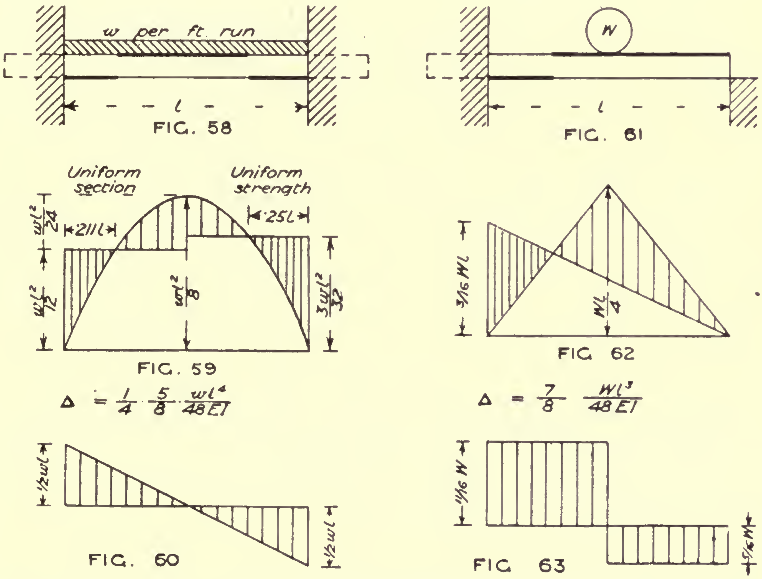


FIG. 58.—Fixed beam with uniformly distributed load. FIG. 59.—Bending moment diagram for fixed beam with uniformly distributed load. FIG. 60.—Shear diagram for fixed beam with uniformly distributed load. FIG. 61.—Beam fixed one end and supported the other with concentrated load. FIG. 62.—Bending moment diagram for beam fixed one end and supported the other with concentrated load. FIG. 63.—Shear diagram for beam fixed one end and supported the other with concentrated load.

form strength, as shown on the two sides of Fig. 59. The fundamental parabola is first drawn and then the top cut off by a rectangle according to the circumstances. Uniform section occurs in rolled joists and practically in reinforced concrete beams; uniform strength occurs in built-up girders where the sectional area is proportional to the stresses. The shear diagram is shown in Fig. 60. It should be specially noted that in the case of reinforced concrete beams loaded and fixed in this manner the bending moment in the centre should be taken as $\frac{wl^2}{12}$ instead of $\frac{wl^2}{24}$, to allow for possible defect in the fixing of the ends. Floor slabs will be dealt with presently, as there are several variations from pure theory, to allow for the practical conditions arising in reinforced concrete construction.

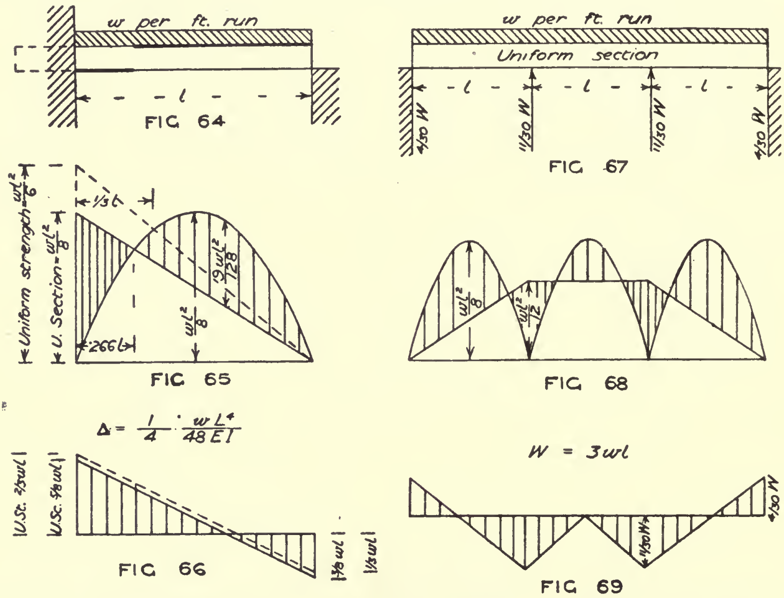


Fig. 64.—Beam fixed one end, supported the other with uniformly distributed load. Fig. 65.—Bending moment diagram for beam fixed one end, supported the other with uniformly distributed load. Fig. 66.—Shear diagram for beam fixed one end, supported the other with uniformly distributed load. Fig. 67.—Continuous beam over three equal spans with uniformly distributed load. Fig. 68.—Bending moment diagram for continuous beam over three equal spans with uniformly distributed load. Fig. 69.—Shear diagram for continuous beam over three equal spans with uniformly distributed load.

When a beam is fixed at one end and merely supported at the other, with a concentrated load in the centre, as in Fig. 61, the bending moment diagram is formed from the fundamental triangle cut off by another triangle, having a height at the fixed end of $\frac{3}{16}$ Wl, as shown in Fig. 62. The distribution of the compression is as shown by the thickened lines in the elevation. The shear diagram will be as Fig. 63.

With a beam similarly supported and carrying a uniformly distributed load, as Fig. 64, the bending moment diagram will depend upon whether the beam has uniform section or uniform strength, as shown in Fig. 65. The shear diagram will also depend upon the same conditions, as shown in Fig. 66. A modification is adopted in the case of reinforced concrete beams supported in this manner, the negative bending moment at the

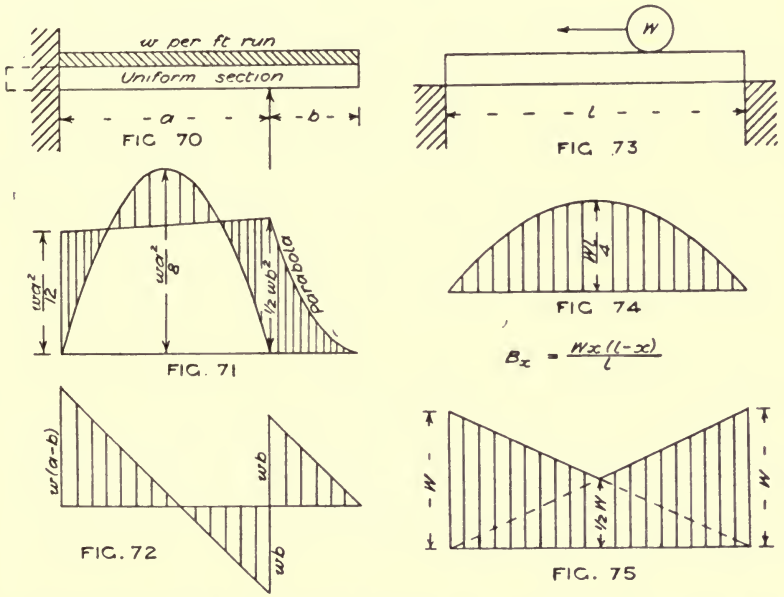


Fig. 70.—Cantilever with column support and uniformly distributed load. Fig. 71.—Bending moment diagram for cantilever with column support and uniformly distributed load. Fig. 72.—Shear diagram for cantilever with column support and uniformly distributed load. Fig. 73.—Beam with concentrated rolling load. Fig. 74.—Bending moment diagram for beam with concentrated rolling load. Fig. 75.—Shear diagram for beam with concentrated rolling load.

fixed end being taken as $\frac{1}{8}wl^2$ and the maximum positive bending moment beyond the centre as $\frac{1}{10}wl^2$ at $\frac{3}{8}l$ from the supported end.

A beam carrying a distributed load, supported at the ends and with a central support also, will have bending moment and shear diagrams for the right-hand span like Fig. 65, and the left hand span like Fig. 65 reversed. The shear diagram will be found in the same way from Fig. 66.

When a beam with a distributed load is continuous over three equal spans, as in Fig. 67, the bending moment diagram will be as Fig. 68 and the shear diagram as Fig. 69. The reactions, or loads on the supports, will be as marked in the shear diagram, the maximum shear always occurring at the supports.

There are many other cases that might be taken of irregular loading or unequal spans, but the foregoing will show the general principles upon which the diagrams should be constructed.

Fig. 70 shows a beam in the condition of a gallery cantilever one end fixed and the other projecting over a column or other support. The bending moment diagram will be as Fig. 71 and the shear diagram as Fig. 72.

All the previous examples of beams with concentrated loads might have diagrams given for the same loads rolling from end to end, but one illustration of the effect of this form of loading must suffice.

Fig. 73 shows a beam supported at the ends with a concentrated load W rolling along it from end to end. The stresses for any given position of the load may be determined by Figs. 23, 74, and 75, but it will be seen that the maximum effect of a stationary concentrated load, which occurs when it is in the centre, is a series of bending moments as ordinates to a triangle of a height $= \frac{Wl}{4}$, while the ordinates must be taken to a parabola of the same height to cover the whole effect of a similar rolling load, as shown in Fig. 74. The shear stress will be increased to double the amount $= W$ at the supports, but at the centre it remains $\frac{1}{2}W$ as shown in Fig. 75. When the load is carried on two wheels the effect will be somewhat less, but no great error will be introduced by assuming the whole load to be concentrated at its centre of gravity. With a continuous beam similar alterations of stress will occur, but the details are rather complicated, and with beams of uniform section, such as rolled joists and reinforced concrete beams, only the additional shear need be considered.

In all beams the span l should be the effective span, that is, from centre to centre of the bearing surfaces, although in the case of rolled joists architects usually take l as the clear span between supports. In reinforced concrete beams the effective span must be taken, which should not exceed twenty-four times the effective depth; and the end spans of continuous beams or slabs must not be considered as fixed at the extreme ends.

For floor slabs in reinforced concrete, supported at the edges, the span is to be taken as the clear span plus thickness of slab, and when continuous over more than one span the distance from centre to centre of the beams. The bending moment across the centre of a square slab supported on four edges and reinforced in two directions at right angles to each other with load uniformly distributed is to be taken $= \frac{Wl}{16}$. The bending moment along the edge of a square slab fixed along four edges, and reinforced in two directions at right angles to each other, with reinforcements bent up over the supports, the load being uniformly distributed, is to be taken $= \frac{Wl}{24}$. For any other ratios of length to breadth, the bending moment on floor slabs should be taken as follows:—

Bending moments on floor slabs, edges supported only, $l =$ length, $b =$ breadth, $W =$ weight of and on slab.

For shorter span	For longer span
$B_c = \frac{Wb}{8} \times \frac{1}{1 + \frac{b^4}{l^4}}$	$\frac{Wl}{8} \times \frac{1}{1 + \frac{l^4}{b^4}}.$

Bending moment on slabs continuous or with fixed edges.

$B_c = \frac{Wb}{24} \times \frac{1}{1 + \frac{b^4}{l^4}}$	$B_e = -\frac{Wb}{12} \times \frac{1}{1 + \frac{b^4}{l^4}}$
$\frac{Wl}{24} \times \frac{1}{1 + \frac{l^4}{b^4}}$	$-\frac{Wl}{12} \times \frac{1}{1 + \frac{l^4}{b^4}}.$

Bending moments on outer bays of floor slabs.

One supported edge, three fixed edges

l in direction of supported edge

$$B_c = \frac{Wl}{10} \times \frac{1}{1 + \frac{b^4}{l^4}}$$

at fixed edge

$$B_e = -\frac{Wl}{8} \times \frac{1}{1 + \frac{b^4}{l^4}}$$

l in direction of fixed edges, same as fixed all round.

Bending moments at corner bays same as last equations above.

When the length of a slab exceeds twice the breadth no calculation is required in the direction of the length.

CHAPTER III.

MOMENTS OF RESISTANCE. LOADS AND REINFORCEMENT.

THE moment of resistance of a beam will depend upon the nature and strength of the material, and the form of cross-section or the disposition of the material with regard to the centre of gravity of the section. A rectangular beam of timber will be the simplest case that can be taken to illustrate the principles, see Fig. 76. Draw two diagonals by joining the opposite corners and put shading lines across the middle triangles. Then remembering that the longitudinal stress is greatest at the top and bottom surfaces, and reduces to nothing at the neutral axis, the shaded portion may be looked upon as giving, by its horizontal width at each part, the comparative intensity of stress; or it may be looked upon as giving the amount of material in use if it were all equally stressed; with this in view it is sometimes called the inertia area, or the area of effective resistance. The "moment of inertia" is the summation of the areas of all the individual fibres or parts in the cross-section of a beam multiplied by the squares of their distances from the neutral axis, $I = \Sigma ay^2$. In Fig. 76, the moment of inertia of the section is $DAG + dag$, where D and d are the depth above and below the neutral axis, A and a the area of each shaded portion, G and g the distance of the centre of gravity of each from the neutral axis. It will be seen that $DAG + dag = 2DAG$, and using b and d for the breadth and depth of the beam, the moment of inertia may be given in terms of the dimensions, then

$$2DAG = 2 \times \tfrac{1}{2}d \times \tfrac{1}{4}bd \times \tfrac{2}{3}(\tfrac{1}{2}d),$$

and multiplying this out and cancelling it will be found that $I = \frac{bd^3}{12}$, which is the moment of inertia for any homogeneous solid rectangular section. If the moment of inertia of such a section be divided by the depth from the neutral axis to the top or bottom, which measurement is generally known as y, it will give the "section modulus" Z, or $\frac{I}{y} = Z$. But $y = \frac{1}{2}d$, therefore $Z = \frac{bd^3}{12 \times \frac{1}{2}d} = \frac{1}{6}bd^2$. The section modulus may be described as the resistance-value of the section depending upon the area and disposition of its parts.

The maximum stress at the upper and lower surfaces is known as "the extreme fibre stress". When a beam is tested to destruction this extreme fibre stress is theoretically the ultimate stress in tension, or compression but it does not cor-

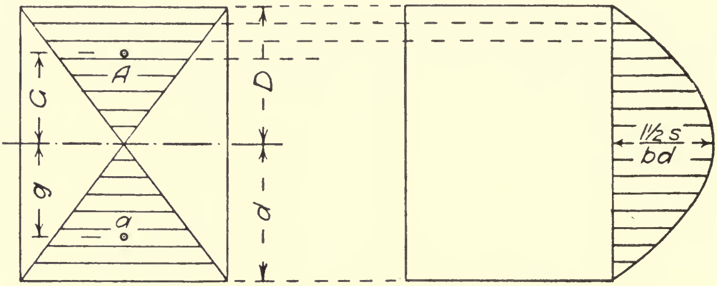


FIG 76 FIG. 77

FIG. 76.—Distribution of longitudinal stress in cross-section of rectangular beam.
FIG. 77.—Distribution of shear stress in cross-section of rectangular beam.

respond with either, as is easily shown by calculation of the stress, and its value is found to vary with the form of cross-section of the material. Several theories have been propounded in explanation of this peculiarity. Professor Rankine suggested one cause to be the fact that the resistance of a material to direct stress is increased by preventing or diminishing the alteration of its transverse dimensions; he also suggested that when a bar of metal is torn asunder the strength indicated is that of the centre part, which is the weakest, whilst when it is broken transversely the strength indicated is that of the outer part which is the strongest. In the case of timber it is suggested that the lateral adhesion of the fibres prevents the outer ones from moving freely, and hence in all cases the actual extreme stress is considerably less than it appears by calculation. This also holds good when the beam is merely subjected to its working load. In any event, the difference really exists, and instead of determining the modulus of rupture from the tensile and compressive strength, it can only be found accurately by experiments on cross-breaking. The extreme fibre stress, calculated from the load required to break any beam, may be stated as so much in excess of the tensile strength, e.g. in plain cement concrete beams the apparent increase in strength under transverse load is such that the extreme fibre stress, or strength modulus, is about $1\frac{3}{5}$ times the tensile strength. The "modulus of rupture for transverse strength," or "strength modulus" is C in the formula $\frac{Wl}{4} = ZC$ (W being in lb. and l in in.; while c in the formula $W = \frac{cbd^2}{L}$ (W being in cwts., b and d in in., and L in ft.) may be called "the coefficient of transverse strength". In general $C = 18c \times 112$; C is the so-called extreme fibre stress = K of Molesworth, k of Tredgold, and f of other writers, and c is the weight in cwts. in centre required to fracture a bar 1 in. square and 1 ft. between the supports.

The relationship of the bending moment to the moment of resistance in a rectangular beam is shown by the following equations:—

Bending moment = Moment of resistance

$$\frac{Wl}{4} = \frac{I}{y}C$$

$$\text{or } \frac{Wl}{4} = ZC$$

$$\text{or } \frac{Wl}{4} = \tfrac{1}{6}bd^2C.$$

If the principle of leverage be considered, the relationship of bending moment and moment of resistance can be shown more clearly in diagrammatic form. The two sets of forces, as shown in Fig. 78, form two "couples," a couple consisting always of two equal parallel forces, and the value of a couple in mechanics is the product of one of the forces into the distance between the pair. The reaction at the support, and the half load in centre transmitted through the beam to that support, form the bending moment couple, and the resistance to tension and compression acting at the centre of gravity of each of the inertia areas form the moment of resistance couple. Thus

$$\tfrac{1}{2}\mathrm{W} \times \tfrac{1}{2}l = \tfrac{1}{4}bd \times \mathrm{C} \times \tfrac{2}{3}d$$

$$\text{whence } \frac{\mathrm{W}l}{4} = \tfrac{1}{6}bd^2\mathrm{C}$$

as previously shown. The shaded portion of the section, previously called the inertia area, is sometimes known as "the modulus figure," as it is the graphic representation of the section modulus Z.

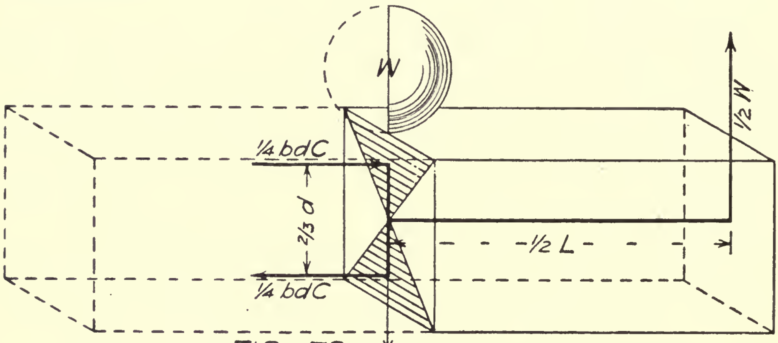


FIG 78

FIG. 78.—Diagram showing the principle of leverage applied to the bending moment and moment of resistance of a beam.

The shear diagrams already shown, give, by their ordinates, the whole shear across any section, but the shear stress is not distributed equally throughout the section. Fig. 77 shows the actual distribution throughout a rectangular section. If the horizontal lines in Fig. 76 be looked upon as dividing up the shaded portion into individual parts, then the shear ordinate in Fig. 77 opposite the bottom of the first part, will be equal to the area of that part; the next shear ordinate will be equal to the combined areas of the first and second part, and so on to the neutral axis. Below the neutral axis the area of each part will be deducted, so that the shear stress is zero at top and bottom of the section, giving a parabola for the complete outline. It can be proved mathematically that the maximum shear at the centre of the section is $1\frac{1}{2}$ times the mean shear, that is, the total shear across the section being s, the mean shear will be $\frac{s}{bd}$, and the maximum shear per square inch will be $\frac{1\frac{1}{2}s}{bd}$. It will thus be seen that the longitudinal stresses are at a maximum at the top and bottom surfaces, where the shear is zero, and that the longitudinal stresses gradually reduce to zero at the neutral axis, while the shear increases to its maximum, at that layer, and that the whole of the fibres are thus doing approximately uniform work.

These matters have been gone into fully for a homogeneous rectangular beam, as a knowledge so gained will be useful in understanding the more complex problem of a reinforced concrete beam.

Loads on Structures.—The first preliminary in finding the actual stresses is to determine the weight of the parts and the loads to be carried, or as they are sometimes called, the structural loads and the superimposed loads, the latter being subdivided into dead loads and live loads. It frequently happens that the structural load cannot be accurately determined until the design is somewhat advanced; in that case reinforced concrete is assumed to weigh 150 lb. per cubic foot, calculated upon an approximate design. Asphalte covering to concrete floor = 12·6 lb. per foot super per inch thick. The weight of wood coverings will depend upon the arrangement, but will average about 5 lb. per foot super.

SUPERIMPOSED LOADS ON FLOORS.

	lb. per sq. ft.
Warehouse (minimum)	224
Book store at library	224
Drillroom or ballroom	150
Public assembly or concert-room, theatre, reading-room at library, workshop or retail shop	112
School or college classroom, office or counting house	100
Hospital ward, lodging-house or hotel bedroom, workhouse, or dwelling-house . . .	70

But in any case, if the actual load to be carried exceeds any of the above, such load must be provided for.

In order that the calculated loads may not be exceeded, every building of the warehouse class must permanently exhibit in a conspicuous position upon each storey a notice stating the maximum superimposed load which may be carried upon the floor of such storey or any part thereof.

A suddenly applied load will produce double the bending moment of a dead load or one gradually applied, while if a load is dropped on to a beam there will be additional energy to be resisted, due to the falling load. In ordinary structures a moving load is not considered to give any increase of stress unless the application of it is rhythmical, as in the case of a drill hall or ballroom, and the higher figures given in the table are for the purpose of making the required allowance for safety. A dense crowd of persons will not generally exceed a load of 84 lb. = $\frac{3}{4}$ cwt. per foot super, and they will then be packed too closely to permit of any movement.

Load from Wind.—For a roof of more than 20 degrees pitch the external load including wind must be estimated at 28 lb. per square foot of sloping surface perpendicular to that surface.

For all other roofs the external load must be estimated at 56 lb. per square foot measured on a horizontal plane, but for a flat roof liable to be used as a store, or otherwise, such additional load as may be probable must be allowed for.

All buildings must be designed to resist safely a wind pressure in any horizontal direction of not less than 30 lb. per square foot of the upper two-thirds of the surface of such building exposed to wind pressure. In exposed positions it may be necessary to allow as much as 56 lb. per square foot.

Loads on Walls, Pillars, and Foundations.—In buildings of more than two storeys in height, except in buildings of the warehouse class, the superimposed loads for the roof and topmost storey must be calculated in full, but for the lower storeys a reduction may be allowed as follows. For the storey next below the topmost storey a reduction of 10 per cent, and for each succeeding storey below a further reduction of 10 per cent, provided that the maximum reduction for any storey shall not exceed 50 per cent of the full load for that storey.

The modulus of elasticity for stone or gravel concrete in cement, not weaker than 1 : 2 : 4 (that is, 1 part by measure of standard Portland cement, 90 lb. being taken as equivalent to a cubic foot, 2 parts clean sand, and 4 parts broken aggregate varying in size from $\frac{1}{4}$ in. to $\frac{3}{4}$ in.) may be treated as constant and taken at one-fifteenth of the modulus of elasticity of mild steel. The actual figures are, in tension or compression :—

	lb. per sq. in.
For concrete, $E_c =$	2,000,000
,, mild steel, $E_s =$	30,000,000

$$\text{Modular ratio } m = \frac{E_s}{E_c} = 15$$

whence it follows that at any given distance from the neutral axis, the stress per square inch on steel will be fifteen times as great as on concrete. The resistance of concrete to tension is neglected, and the steel reinforcement is assumed to carry all the tension. The stress on the steel reinforcement is taken as uniform on a cross-section, and that on the concrete as uniformly varying.

The ultimate compressive strength of the concrete should not be less than is given in the following table :—

Uses.	Proportion by volume.			Ultimate Compressive Resistance in lb. per sq. in.	
	Cement.	Sand.	Coarse Material.	28 days after Mixing.	90 days after Mixing.
For beams and pillars .	1	2	4	1800	2400
,, ,, ,, ,, .	1	$1\frac{1}{2}$	3	2100	2800
,, pillars only . .	1	1	2	2700	3600

Working Stresses.—If the concrete is of such a quality that its crushing strength is 2400 to 3000 lb. per square inch after twenty-eight days, and the steel has a tenacity of not less than

60,000 lb. per square inch, the following stresses may be allowed :—

	lb. per sq. in.
Concrete in compression in beams subjected to bending	600
Concrete in columns under simple compression	500
Concrete in shear in beams	60
Adhesion of concrete to metal	100
Steel in tension	15,000 to 17,000

When the proportions of the concrete differ from those stated above the stress in compression allowed in beams may be taken at one-fourth, and that in columns at one-fifth of the crushing strength of cubes of the concrete of sufficient size at twenty-eight days after gauging. If stronger steel is used than stated above, the allowable tensile stress may be taken at one-half the stress at the yield point of the steel.

The working stresses may be tabulated as follows :—

Except as provided for in pillars, the safe working stresses on concrete should not exceed the following :—

Stresses on Concrete.	Proportions by volume.					
	Cement. 1	Sand. 2	Coarse Material. 4	Cement. 1	Sand. ½	Coarse Material. 3
	Stress in lb. per sq. in.			Stress in lb. per sq. in.		
Direct compressive stress	600			700		
Extreme fibre stress .	600			700		
Frictional stress between concrete and steel .	60			60		
Shearing stress . .	60			60		
Tensile stress . . .	nil			nil		

and the safe working stresses on mild steel should not exceed the following :—

Stresses on Mild Steel.	lb. per sq. in.
Direct compression stress = mc, where m = modular ratio and c = compressive stress on concrete surrounding the steel	mc or 16,000 whichever is the lesser.
Tensile stress on mild steel . . .	16,000
Shearing ,, ,, ,, ,, . . .	12,000

In calculating the steel reinforcement to provide for the shear stresses, only the balance after allowing 60 lb. per square inch on the section of concrete need be provided for.

In order that a reinforcing rod may take the direct stress at any point for which it is calculated, the "grip" length in inches of a bar embedded in concrete, measured along the bar from any given cross-section to the end of the bar, must not be less than $\frac{10 \times \text{sectional area} \times t}{\text{perimeter}}$ which gives for rounds and squares $2500td$, and for flats $\frac{5600td}{b+d}$ where t = the tensile or compressive stress at the given cross-section, and d = the diameter of the bar in inches.

In designing the reinforcement the following rules must be observed. The least diameter or thickness of the main reinforcing bars in beams or slabs must not be less than $\frac{1}{4}$ in., but wiring may be used solely for holding the bars in position during the process of placing the concrete. All other reinforcements must be at least $\frac{1}{8}$ in. diameter or other section of equal area. The reinforcement must not be placed nearer the face of the concrete than $\frac{1}{2}$ in. in slabs, 1 in. in cross beams, and $1\frac{1}{2}$ in. in main beams and pillars. A distance of at least 1 in. must be left horizontally between the bars and $\frac{1}{2}$ in. vertically except at points where the bars are in direct contact and transverse to one another. The maximum distance between the main tensile reinforcement of slabs must not be greater than 12 in.

Mesh reinforcement must be of such dimensions as will enable the coarse material in the concrete to pass easily through the meshes of such reinforcement. Where compressive reinforcement is provided in beams and calculated to take part of the compression it must be anchored by bars extending down to the bottom of the middle third of the effective depth of the beam. The anchors must not be further apart, centre to centre, than the effective depth of the beam.

All beams must be provided with shear members throughout the length of the beam. All shear members must be spaced according to the actual shear stress or at a distance apart not exceeding the effective depth of the beams, and extend from the centre of the tensile reinforcement to the centre of pressure of the concrete under compression, be passed under, or round, or secured to, the tensile reinforcement, and have a mechanical anchorage with the concrete at the free ends or throughout their length.

The brackets or splays at the ends of beams and fillets in the angles between floor slabs and beams are necessary for securing efficiency and stiffness of the work, but are not to be taken into account as reducing the span.

The effective depth of a beam or slab is measured from the compressional edge of the concrete to the centre of the tensional reinforcement.

Tee Beams.—There is a modification sometimes permitted in calculating the strength of secondary beams. A portion of the floor slab not exceeding twelve times the thickness of the slab plus the width of the rib may be taken as part of the beam in computing the bending moment of the beam, which is then known as a tee beam. In that case the tensile reinforcement of the slab must be continuous across the full width of the portion of the slab forming the flange of the tee beam, and shear reinforcement is required in the plane of junction of the rib and the flange.

Reinforced Concrete Pillars.—In reinforced concrete pillars the maximum value of the ratio of length to effective diameter must be taken between the lateral supports, irrespective of any bracketing. The effective diameter will be the measurement to the outside of the vertical reinforcement. Pillars are deemed to have fixed ends when the ends are sufficiently secured to other parts of the construction having such rigidity as will maintain the axis of the pillar at the ends in its original vertical position under all loads less than the crippling load.

When both ends are fixed the following stresses may be allowed:—

$\frac{\text{Length}}{\text{Effective Diam.}} =$	Stress Allowed.
18	Full stress
22	·8 of ,,
24	·6 ,, ,,
27	·4 ,, ,,
30	·2 ,, ,,

If P = the maximum pressure on pillars and compression members having fixed ends, compression members not having both ends fixed may have working loads as follows:—

One end fixed and one hinged . . . . . . $= \frac{1}{2}$ P
Both ends hinged . . . . $= \frac{1}{4}$ P
One end fixed, the other free or not supported in all directions . . . . $= \frac{1}{16}$ P

Each pillar with straight laterals (hooping) must have at least four lines, and with curved laterals six lines, of vertical reinforcement throughout its entire length.

The least diameter of straight laterals must be not less than $\frac{3}{16}$ in., and curved laterals $\frac{1}{8}$ in. The pitch of the laterals must not exceed $\frac{6}{10}$ of the effective diameter of the pillars, joints in the vertical reinforcement may only be made at a floor level or other point of lateral support. At all joints in the vertical reinforcement, the bars should overlap a distance equal to required grip length, or have flitch bars of double that length, or have butt ends and a close fitting sleeve.

In the case of rectangular pillars in which the ratio between the greater and lesser diameter exceeds $1\frac{1}{2}$, the cross-section of the pillar must be subdivided by cross ties, and the number of vertical rods must be such that the distance between the rods along the longer side of the rectangle shall not exceed the distance between the rods along the shorter side of the rectangle. The total cross-sectional area of the vertical reinforcement in any pillar must not be less than 0·8 per cent of the area of the hooped core, and the volume of the lateral reinforcement must not be less then 0·5 per cent of the volume of the hooped core.

Reinforced Concrete Walls.—Reinforced concrete enclosure walls, where the main loads of the structure are carried by pillars to the foundations, may be not less than 6 in. thick provided they are designed and reinforced under similar rules to the remainder of the structure for the loads and pressures they may have to carry. When portions of the external walls between the reinforced concrete pillars and beams are constructed of other materials than reinforced concrete, they must be not less than 8½ in. thick for the top 20 feet of their height, and 13 in. for the remainder of their height. All walls must be securely connected to the continuous part of the reinforced concrete construction. Party walls of reinforced concrete must nowhere be less than 8 in. thick.

Openings may be made in external walls provided that in any storey above the ground storey the aggregate area of such openings will not exceed three-fourths of the whole area, or the aggregate widths nine-tenths of the whole length of such storey.

[The above notes are based upon the probable conclusions which will be put forward by the London County Council to regulate the construction of reinforced concrete buildings, but as the matter is still *sub judice* no authoritative statements can be made.]

CHAPTER IV.

NOTATION, FORMULÆ, AND EXAMPLES.

Resistance Moments for Rectangular Beams.—*Notation.*

b = *breadth* of beam in inches.

d = the effective *depth* of the beam in inches, i.e. the distance from the top of the beam to the common centre of gravity of the tensile reinforcement.

n = the distance of the *neutral* axis from the compressed edge of the beam in inches.

k = the fraction of the depth given by the distance of the neutral axis from the compressive edge $= \frac{n}{d}$.

A_c = *area* of *concrete* in square inches $= bd$.

A_t = *area* of *tensile* reinforcement (in square inches).

E_s = *elastic* modulus of *steel* in tension.

E_c = *elastic* modulus for *concrete* in compression.

$m = \frac{E_s}{E_c} = 15$ = *Modular* ratio.

B = *bending* moment of the external loads and forces in lb.-in.

R = *resistance* moment of the internal stresses in the beam in lb.-in.

c = *compressive* safe working stress on extreme edge of the concrete in compression, i.e. the strength modulus for concrete in compression (in pounds per square inch).

t = *tensile* working stress on steel in tensile reinforcement (in pounds per square inch of cross-section).

r = ratio of A_t to bd, i.e. $r = \frac{A_t}{bd}$.

p = the *percentage* of tensile reinforcement $= 100r$.

4 *

a = the *arm* of the couple formed by the compressive and tensile forces in the beam.

W = the total *weight* to be carried by a beam.

l = the *length* of the effective span of a beam.

Fig. 79 shows the cross-section of a reinforced concrete beam and Fig. 80 a diagram illustrating the use of the terms given above.

Neutral Axis.—In a homogeneous beam the stresses are proportional to the distances from the neutral axis, but in a discrete beam, such as a beam of concrete reinforced with steel, on

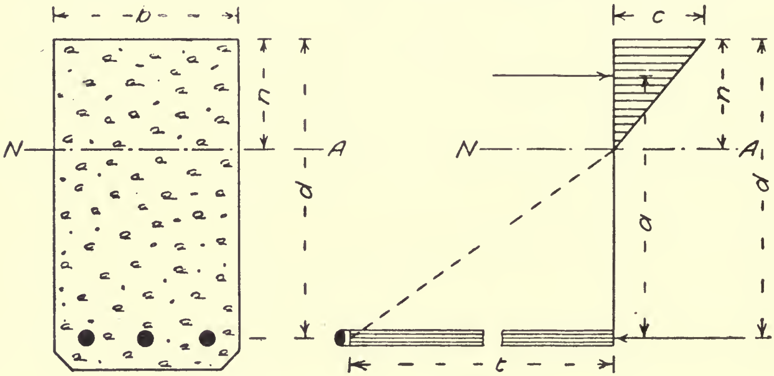


FIG. 79.—Cross-section of a reinforced concrete beam. FIG. 80.—Diagram showing neutral axis and resistance areas of a reinforced concrete beam.

account of the greater rigidity of the steel, at a given distance from the neutral axis the stress in the steel will be m times as great as in the concrete.

Calculations for locating the neutral axis are based on the equation

$$\frac{n}{d} = \sqrt{(r^2m^2 + 2rm)} - rm$$

and

$$n = [\sqrt{(r^2m^2 + 2rm)} - rm]d.$$

Mean compressive stress on the concrete $= \frac{c}{2}$.

Arm of resistance moment $a = d - \frac{n}{3}$.

Tensile resistance moment $R_t = tA_t\left(d - \frac{n}{3}\right)$.

Compressive resistance moment $R_c = \frac{c}{2}bn\left(d - \frac{n}{3}\right)$.

Maximum bending moment under distributed load ends supported $B_c = \frac{Wl}{8}$.

Maximum tensile stress $t = \frac{B}{rbd^2(1 - \frac{1}{3}k)}$.

Maximum compressive stress $c = \frac{2B}{kbd^2(1 - \frac{1}{3}k)}$.

Example of designing a Reinforced Concrete Beam.

A reinforced concrete beam is required to carry a uniformly distributed load of 8 tons in addition to its own weight over an effective span of 16 ft. The stress on the steel not to exceed 16,000 lb. per sq. in. and on the concrete 600 lb. per sq. in.

Assume width of beam in inches to be $\frac{3}{4}$ of span in feet, $\frac{3}{4} \times 16 = 12$ in. and total depth to be twice width $= 24$ in. Approximate weight of beam =

$$\frac{12 \times 24}{12 \times 12} \times 150 = 300 \text{ lb.}$$

$$\text{Total load } 8 \times 2240 + 300 = 18220 \text{ lb.}$$

Maximum bending moment $= \frac{Wl}{8} = \frac{18220 \times 16 \times 12}{8} = 437280$ lb.-in.

Then $m\left(\frac{c}{t}\right) = \frac{k}{1-k} = 15\left(\frac{600}{16000}\right) = \cdot 5625$

and $k = \cdot 5625(1-k) = \frac{\cdot 5625}{1 \cdot 5625} = \cdot 36$

but $ck = 2rt$, or $600 \times \cdot 36 = 2r \times 16000$.

$$\therefore\ r = \frac{600 \times \cdot 36}{16000 \times 2} = \cdot 00675$$

$$t = \frac{B}{rbd^2(1 - \frac{1}{3}k)} = \frac{437280}{\cdot 00675 \times 12 \times d^2 \times (1 - \frac{1}{3} \times \cdot 36)} = 16000$$

$$\therefore\ d = \sqrt{\frac{437280}{\cdot 00675 \times 12 \times \cdot 88 \times 16000}} = 19 \cdot 5 \text{ in.}$$

Sectional area of metal $= rbd$

$$= \cdot 00675 \times 12 \times 19 \cdot 5 = 1 \cdot 58 \text{ sq. in.}$$

say four $\frac{3}{4}$ in. rods.

$$t = \frac{B}{rbd^2(1 - \frac{1}{3}k)} = \frac{437280}{\cdot 00675 \times 12 \times 19 \cdot 5^2 \times \cdot 88} = 16133 \text{ lb. per sq. in.}$$

$$c = \frac{2B}{kbd^2(1 - \frac{1}{3}k)} = \frac{2 \times 437280}{\cdot 5625 \times 12 \times 19 \cdot 5^2 (\cdot 88)} = 387 \text{ lb. per sq. in.}$$

Obtaining depth by $c = 600$

$$600 = \frac{2 \times 437280}{\cdot 5625 \times 12 \times d^2 \times \cdot 88}$$

$$d = \frac{2 \times 437280}{600 \times \cdot 5625 \times 12 \times \cdot 88} = 15 \cdot 6 \text{ in.}$$

It therefore appears to be impossible to stress both materials to their maximum value, but the most economical percentage of reinforcement $= 0 \cdot 675$ per cent.

With a fixed ratio between the working stresses of steel and concrete, say 17000 lb. per square inch and 600 lb. per square inch, and a given percentage of reinforcement, beams may be calculated approximately by the formula

$$B_c = Cbd^2$$

where for		·5	per cent.	C	= 76
,,	,,	·75	,,	,,	= 98
,,	,,	1·0	,,	,,	= 108
,,	,,	1·25	,,	,,	= 115
,,	,,	1·5	,,	,,	= 122

Thus for a reinforced concrete beam as above 12 in. by 19·5 in. with ·75 per cent reinforcement

$$cbd^2 = 98 \times 12 \times 19 \cdot 5^2 = 447174 \text{ lb.-in.,}$$

which is rather higher than the bending moment required because a higher percentage of reinforcement has been taken.

By the authors' empirical formula for safe distributed load

$$W_{cwts} = (\cdot 37p + \cdot 214) \frac{bd^2}{L}$$

$$= (\cdot 37 \times \cdot 675 + \cdot 214) \frac{12 \times 19 \cdot 5^2}{16}$$

$$= 132 \text{ cwt.} = 6 \cdot 6 \text{ tons}$$

as against 8 tons for which the beam has been designed, but an approximate formula should always be on the safe side.

Example of Working when the Dimensions are given :—

say $b=10$, $d=20$, $bd=A_c=200$

say 1 per cent reinforcement

$$A_t=\frac{1}{100}\times 200=2 \text{ sq. in.}$$

$$r=\frac{A_t}{A_c}=\frac{2}{200}=\frac{1}{100}=\cdot 01$$

$$m=15$$

$$\begin{aligned} n &= [\sqrt{(\cdot 01^2\times 15^2+2\times\cdot 01\times 15)}-\cdot 01\times 15]20 \\ &= [\sqrt{(\cdot 0225+\cdot 3)}-\cdot 15]20 \\ &= (\cdot 5679-\cdot 15)20 \\ &= \cdot 4179\times 20=8\cdot 358 \text{ in.} \end{aligned}$$

$$\begin{aligned} R_t &= 16000\times 2\left(20-\frac{8\cdot 358}{3}\right) \\ &= 32000\times 17\cdot 21\dot{3} \\ &= 550827 \text{ lb.-in.} \end{aligned}$$

$$\begin{aligned} R_c &= \frac{600}{2}\times 10\times 8\cdot 358\ (17\cdot 21\dot{3}) \\ &= 25074\times 17\cdot 21\dot{3} \\ &= 431607 \text{ lb.-in.} \end{aligned}$$

If the span be 20 ft. the total load distributed based upon the resistance moment of the concrete will be

$$B=\frac{Wl}{8}=R \ \therefore\ W=\frac{8R}{l}=\frac{8\times 431607}{20\times 12}=14386\cdot 9 \text{ lb.}$$

Now by R.I.B.A. Journal, 1907, p. 533, Report of Joint Committee on Reinforced Concrete—

$$p=r-\frac{A_t}{A_c}=\cdot 01$$

$$k=\frac{n}{d}=\frac{8\cdot 358}{20}=\cdot 4179$$

$$1-\tfrac{1}{3}k=1-\tfrac{1}{3}\times\cdot 4179=\cdot 8607$$

$$t=\frac{431607}{\cdot 01\times 10\times 20^2(\cdot 8607)}=12{,}246 \text{ lb. per sq. in.}$$

$$c=\frac{2\times 431607}{\cdot 4179\times 10\times 20^2\times\cdot 8607}=600 \text{ lb. per sq. in.}$$

It is evident then that the steel is in excess for the maximum economy. Try a reduced percentage of reinforcement.

Say $b=10$, $d=20$, $bd=A_c=200$,

say $\frac{3}{4}$ per cent ($\cdot 75$) reinforcement.

$$A_t=\frac{\cdot 75}{100}\times 200=1\cdot 5 \text{ sq. in.}$$

$$r=\frac{A_t}{A_c}=\frac{1\cdot 5}{200}=\cdot 0075$$

$$m=15$$

$$n=[\sqrt{(\cdot 0075^2\times 15^2+1\cdot 5\times\cdot 0075\times 15)}-\cdot 0075\times 15]20$$
$$=[\sqrt{(\cdot 01265625+\cdot 16875)}-\cdot 1125]20$$
$$=(\cdot 4259-\cdot 1125)20=6\cdot 268 \text{ in.}$$

$$R_t=16000\times 1\cdot 5\left(20-\frac{6\cdot 268}{3}\right)$$
$$=24000\times 17\cdot 91=429840 \text{ lb.-in.}$$

$$R_c=\frac{600}{2}\times 10\times 6\cdot 268\ (17\cdot 91)$$
$$=18804\times 17\cdot 91=336779 \text{ lb.-in.}$$

$$W=\frac{8R}{l}=\frac{8\times 336779}{20\times 12}=11226 \text{ lb.}$$

$$p=r=\cdot 0075$$

$$k=\frac{n}{d}=\frac{6\cdot 268}{20}=\cdot 3134$$

$$1-\tfrac{1}{3}k=\cdot 8955$$

$$t=\frac{336779}{\cdot 0075\times 10\times 20^2\times\cdot 8955}=12536 \text{ lb. per sq. in.}$$

$$c=\frac{2\times 336779}{\cdot 3134\times 10\times 20^2\times\cdot 8955}=599 \text{ lb. per sq. in.}$$

Therefore when the reinforcement is decreased the total safe load is less, the neutral axis is higher, and the steel is stressed nearer to its full value. Try a further reduction in the percentage of reinforcement.

Say $b=10$, $d=20$, $bd=A_c=200$

say $\cdot 5$ per cent reinforcement

$$A_t=\frac{\cdot 5}{100}\times 200=1 \text{ sq. in.}$$

$$r=\frac{A_t}{A_c}=\frac{1}{200}=\cdot 005$$

$$m = 15$$
$$n = [\sqrt{(\cdot005^2 \times 15^2 + 1 \times \cdot005 \times 15)} - \cdot005 \times 15]\, 20$$
$$= [\sqrt{(\cdot005625 + \cdot075)} - \cdot075]\, 20$$
$$= (\cdot2839 - \cdot075)\, 20 = 4\cdot178$$
$$R_t = 16000 \times 1\left(20 - \frac{4\cdot178}{3}\right)$$
$$= 16000 \times 18\cdot607 = 297712 \text{ lb.-in.}$$
$$R_c = \frac{600}{2} \times 10 \times 4\cdot178\ (18\cdot607)$$
$$= 12534 \times 18\cdot607 = 233220 \text{ lb.-in.}$$
$$W = \frac{8R}{l} = \frac{8 \times 233220}{20 \times 12} = 7774 \text{ lb.}$$
$$p = \cdot005$$
$$k = \frac{4\cdot178}{20} = \cdot2089$$
$$1 - \tfrac{1}{3}k = \cdot9304$$
$$t = \frac{233220}{\cdot005 \times 10 \times 20^2 \times \cdot9304} = 12{,}533 \text{ lb. per sq. in.}$$
$$c = \frac{2 \times 233220}{\cdot2089 \times 10 \times 20^2 \times \cdot9304} = 599 \text{ lb. per sq. in.}$$

Shear Stress in Beams.

a_s = total sectional area of shear reinforcement (diagonal tension) on each side of centre.

$$a_s = \frac{bd}{48}$$

placed approximately from centre at, ·25, ·4, ·57, ·72, ·84, ·93, ·98, 1·0 of semi-span.

For example, take the beam 12 in. by 19·5 in. as first above

$$a_s = \frac{bd}{48} = \frac{12 \times 19\cdot5}{48} = 4\cdot875$$

say 8 stirrups in length of ½ span

$$\frac{4\cdot875}{8} = \cdot61 \text{ sq. in. each.}$$

∴ section for 2 stirrup-bars = $\frac{5}{8}$ sq. in. each.

,,	4	,,	,,	= $\frac{7}{16}$	,,
,,	6	,,	,,	= $\frac{3}{8}$	,,
,,	8	,,	,,	= $\frac{5}{16}$	,,

Owing to the reduction of the bending moment towards the supports of a beam under a uniformly distributed load the tensile reinforcement may be bent up to take part in the shear stress as follows: one-third at a distance of $\cdot 21l$ from the centre of bearing or one-half at a distance of $\cdot 15l$. Generally it will depend upon the number of rods which figure is adopted. With three rods one could be turned up at $\cdot 21l$ and with four rods two could be turned up at $\cdot 15l$.

The beam as designed will now be in section and elevation as Figs. 81 and 82 which may be looked upon as typical.

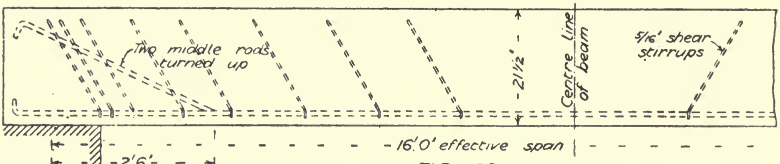


FIG. 82

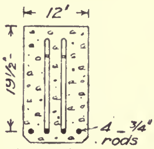


FIG. 81

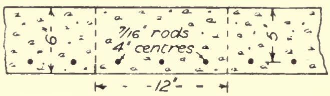


FIG. 83

FIG. 81.—Cross-section of reinforced concrete beam. **FIG. 82.**—Part elevation of beam showing the reinforcement. **FIG. 83.**—Section of reinforced concrete floor slab.

Rectangular Beams with Double Reinforcement.—In addition to the symbols already given, let

x = distance from the top of the beam, to the centre of gravity of the top reinforcement.

A = area of compressive reinforcement in square inches.

c_s = compressive working stress on steel in compressive reinforcement.

Then $$n = \frac{m}{b}(\mathrm{A} + \mathrm{A}_t)\left[\sqrt{1 + \frac{2b(x\mathrm{A} + d\mathrm{A}_t)}{m(\mathrm{A} + \mathrm{A}_t)^2}} - 1\right]$$

$$c = \frac{n\mathrm{B}}{\frac{1}{3}n^3 b + m\{\mathrm{A}(n - x)^2 + \mathrm{A}_t(d - n)^2\}}$$

$$t = cm\frac{d - n}{n}$$

$$c_s = cm\frac{n - x}{n}$$

Example of Stresses in a Floor Slab.

Say a reinforced concrete floor slab 7 ft. 6 in. span, supported at the ends, 5 in. effective thickness, reinforced with $\frac{7}{16}$ in. bars, 4 in. centre to centre, carrying a superimposed load of 2 cwt. per foot super. as in Fig. 83.

Structural load $\frac{(5+1)12}{144} \times 150$	=	75	lb. per sq. ft.
Wood floor, etc. . . .	=	5	,,
Superimposed load . .	=	224	,,
		304	,,

Taking a 12 in. breadth of slab, the span

$$l = (12 \times 7{\cdot}5) + 6 = 96 \text{ in.}$$

and the bending moment in centre

$$B_c = \frac{Wl^2}{8} = \frac{304}{12} \times \frac{96^2}{8} = 29{,}184 \text{ lb.-in.}$$

The effective depth = 5 in., in 12 in. wide there are $\frac{12}{4} = 3$ bars, each $\frac{7}{16} = 0{\cdot}15$ sq. in. area.

$$\text{Then } r = \frac{A_t}{A_c} = \frac{0{\cdot}15 \times 3}{12 \times 5} = {\cdot}0075,\ m = 15$$

$$\begin{aligned} k &= \sqrt{(r^2m^2 + 2rm)} - rm \\ &= \sqrt{({\cdot}0075^2 \times 15^2 + 2 \times {\cdot}0075 \times 15)} - {\cdot}0075 \times 15 \\ &= {\cdot}3750 \end{aligned}$$

$$1 - \tfrac{1}{3}k = 1 - \frac{{\cdot}3750}{3} = {\cdot}875.$$

Stress in steel

$$\begin{aligned} t &= \frac{B_c}{rbd^2(1 - \frac{1}{3}k)} = \frac{29184}{{\cdot}0075 \times 12 \times 5^2({\cdot}875)} \\ &= \frac{29184}{1{\cdot}969} = 14{,}822 \text{ lb. per sq. in.} \end{aligned}$$

Stress in concrete

$$\begin{aligned} c &= \frac{2B_c}{kbd^2(1 - \frac{1}{3}k)} = \frac{2 \times 29184}{{\cdot}3750 \times 12 \times 5^2({\cdot}875)} \\ &= 593 \text{ lb. per sq. in.} \end{aligned}$$

which is satisfactory as being reasonably within the allowable limits of maximum stress.

Resistance Moment for Tee Beams.—A_t = *area* of *tensile* reinforcement.

b = *breadth* of compression flange of tee beam.

d = effective *depth* of the beam, measured from the top of the beam (surface of floor) to the centre of gravity of the tensile reinforcement.

d_s = Total *depth* of *slab*.

p = *percentage* of reinforcement in the equation

$$\frac{A_t}{bd_s} = \frac{p}{100}$$

d' = the ratio of d to d_s, i.e.

$$d' = \frac{d}{d_s}$$

n = the *neutral* axis depth, i.e. the distance from the compressed edge to the *neutral* axis.

k = the ratio of n to d, i.e. the fraction of depth from the compressed edge to the *neutral* axis.

$$k = \frac{n}{d}$$

c = the compressive stress on the compressed edge of the concrete.

z = distance from upper surface to centre of pressure in concrete.

$$= \frac{d_s}{3} \times \frac{3n - 2d_s}{2n - d_s}$$

a = the arm of the compressive and tensile forces.

Tee Beams with Double Reinforcement.—When the neutral axis is above or coincides with the under side of the floor slab the formulæ for doubly reinforced rectangular beams may be used, but b will equal the breadth of compression flange of tee beam instead of breadth of rectangular beam. When the neutral axis is greater than the depth of floor slab

$$n = \frac{\frac{1}{2}bd_s^3 + m(A_t d + Ax)}{bd_s + m(A_t + A)}$$

$$c = \frac{nB}{[\frac{1}{2}bd_s(n - z)(2n - d_s) + m\{A(n - x)^2 + A_t(d - n)^2\}]}$$

$$a = d - z.$$

s' = the slab depth ratio = d_s/d.

Fig. 84 shows the section of a reinforced tee beam and Fig. 85 a diagram illustrating the use of the terms given above.

The position of the neutral axis must be obtained from the equation

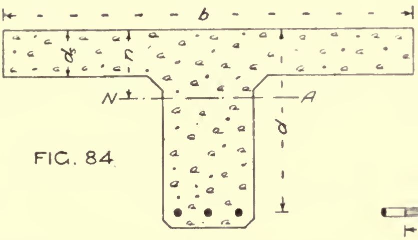
$$k = \frac{0{\cdot}5s' + 0{\cdot}15p}{1 + 0{\cdot}15p}$$

The mean compressive stress must not be taken at more than

$$c - \frac{c}{2kd'}$$

$$\text{or} \qquad c\left[1 - \frac{s'}{2k}\right]$$

$$\text{or} \qquad c\left[1 - \frac{1}{2kd'}\right].$$



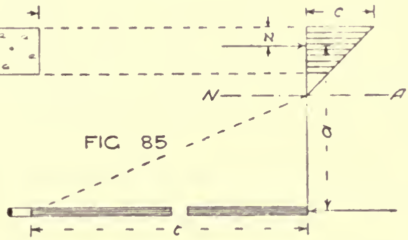


FIG. 84.—Cross-section of reinforced concrete tee beam. FIG. 85.—Diagram of neutral axis and resistance areas.

The compressive resistance moment $= \mathrm{R}_c$ where

$$\mathrm{R}_c = \left(c - \frac{c}{2kd'}\right)(bd_s)a \text{ or}$$

$$c\left[1 - \frac{s'}{2k}\right]bd_s\,a \text{ or}$$

$$c\left[1 - \frac{1}{2kd'}\right]bd_s\,a.$$

Stress on the concrete $= c$ where

$$c = \frac{2\mathrm{B}n}{bd_s(2n - d_s)(d - z)}$$

The tensile resistance moment $= \mathrm{R}_\mathrm{T}$ where

$$\mathrm{R}_\mathrm{T} = t\mathrm{A}_t a.$$

Stress on the steel reinforcement $= t$ where

$$t = \frac{\mathrm{B}}{\mathrm{A}_t(d - z)}.$$

Shearing force at ends of beam

$$S = \frac{wl}{2}.$$

Shearing stress on section of beam

$$= \frac{S}{b(d-z)}.$$

The floor slab to the extent of not more than twelve times the thickness of slab may be taken as the width of compression flange.

Example of Calculation of Tee Beam.—Say the floor is 20 ft. span, the floor slab $4\frac{1}{2}$ in. total thickness, the external load $\frac{3}{4}$ in. asphalte and 140 lb. per foot super uniformly distributed. Beams 12 in. wide and 18 in. deep below concrete to centre of reinforcement, which consists of six $\frac{3}{4}$ in. rods; beams 6 ft. centre to centre, as in Figs. 86 and 87.

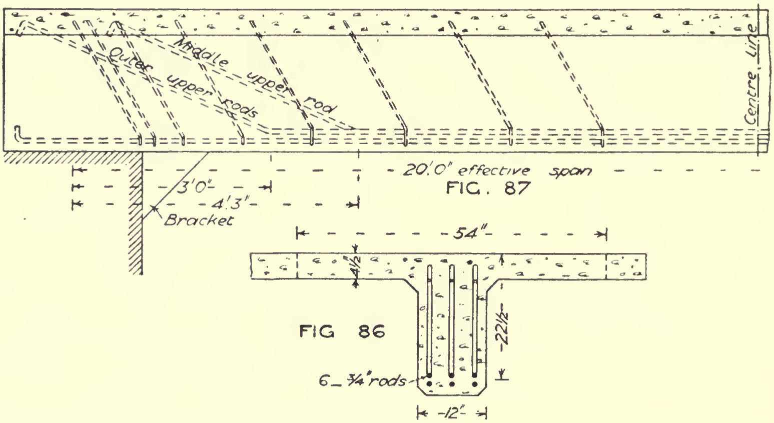


Fig. 86.—Cross-section of reinforced concrete beam and floor slab from calculations. Fig. 87.—Part elevation of the same.

Assume twelve times thickness of slab as width of flange of tee beam.

$12 \times 4{\cdot}5 = 54$ in.

Weight of floor:—

	lb. per ft. run.
Slab $6 \times \frac{4 \cdot 5}{12} \times 150$	$= 337 \cdot 5$
Beam $\frac{(18 + 2 \cdot 5)12 \times 150}{144}$ . . .	$= 256 \cdot 25$
Asphalte $6 \times \cdot 75 \times 12 \cdot 6$ . . .	$= 56 \cdot 7$
Superimposed load 6×140 . .	$= 840 \cdot 0$
	$1490 \cdot 45$

say total load 1490 lb.

Maximum bending moment, ends supported,

$$\frac{wl^2}{8} = \frac{\frac{1490}{12}(20 \times 12)^2}{8} = 894000 \text{ lb.-in.}$$

The sectional area of six $\frac{3}{4}$ rods $= 6 \times \cdot 442 = 2 \cdot 652$.

Width of slab forming flange of tee beam = twelve times thickness,

$$12 \times 4 \cdot 5 = 54 \text{ in.}$$

Position of neutral axis

$$k = \frac{0 \cdot 5s' + 0 \cdot 15p}{1 + 0 \cdot 15p}$$

$$= \frac{0 \cdot 5 \times \frac{4 \cdot 5}{(18 + 4 \cdot 5)} + 0 \cdot 15 \times \frac{2 \cdot 652}{54 \times 4 \cdot 5} \times 100}{1 + 0 \cdot 15 \times \frac{2 \cdot 652}{54 \times 4 \cdot 5} \times 100}$$

$$= 0 \cdot 226$$

$$n = kd = 0 \cdot 266(4 \cdot 5 + 18)$$

$$= 5 \cdot 09.$$

Distance from upper surface to centre of pressure in concrete

$$z = \frac{d_s}{3} \times \frac{3n - 2d_s}{2n - d_s}$$

$$= \frac{4 \cdot 5}{3} \times \frac{3 \times 5 \cdot 09 - 2 \times 4 \cdot 5}{2 \times 5 \cdot 09 - 4 \cdot 5}$$

$$= 1 \cdot 656 \text{ in.}$$

Stress on the steel reinforcement

$$t = \frac{\mathrm{B}}{\mathrm{A}_t(d - z)} = \frac{89400}{2 \cdot 652(22 \cdot 5 - 1 \cdot 656)}$$

$$= 16172 \text{ lb. per sq. in.}$$

Stress on the concrete

$$c=\frac{2Bn}{bd_s(2n-d_s)(d-z)}$$

$$=\frac{2\times 89400\times 5{\cdot}09}{54\times 4{\cdot}5(2\times 5{\cdot}09-4{\cdot}5)(22{\cdot}5-1{\cdot}656)}$$

$$=316 \text{ lb. per sq. in.}$$

Maximum shearing force at ends of beam

$$S=\frac{wl}{2}=\frac{1490{\cdot}45\times 20}{2}=14904{\cdot}5 \text{ lb.}$$

Shearing stress on section of beam

$$=\frac{S}{b(d-z)}=\frac{14904{\cdot}5}{12(22{\cdot}5-1{\cdot}656)}=59{\cdot}6 \text{ lb. per sq. in.}$$

This is within the allowable limit of 60 lb. per square inch, but it is usual to bend up the upper row of steel rods at the ends, so that there will be ample margin.

The perimeter (o) of a $\frac{3}{4}$ in. rod is $\cdot 75\times 3{\cdot}1416=2{\cdot}36$ in.

The adhesion stress of the lower rods near the ends will be

$$\frac{\frac{w}{12}(l-12)}{2o(d-z)}=\frac{\frac{1490}{12}(240-12)}{2\times 3\times 2{\cdot}36(22{\cdot}5-1{\cdot}656)}$$

$$=96 \text{ lb. per sq. in.}$$

which is within the allowable limit of 100 lb. per square inch.

Formulæ for Reinforced Pillars.—

C = the ultimate crushing resistance of concrete at 3 months.

S_F = the safety factor = say 4.

k = the reciprocal of the safety factor $=\frac{1}{S_F}$.

f = form factor, depending upon the form or type of laterals, (see table).

s = spacing factor, depending upon the spacing or pitch of the laterals (see table)

r = ratio of the volume of lateral reinforcement to the volume of the hooped core in any given length of pillar.

The stress on the concrete in the area bounded by the lateral reinforcement must not exceed

$$kC(1+fsr)$$

The value of f and s are given in the following table :—

Form of Lateral Reinforcement.	Form Factor $=f$.	Spacing of Laterals in terms of Diameter of Hooped Core.	Spacing Factor $=s$.
Helical . .	1·0	0·2d	32
,, . .	1·0	0·3d	24
,, . .	1·0	0·4d	16
Circular Hoops .	0·75	0·2d	32
,, ,, .	0·75	0·3d	24
,, ,, .	0·75	0·4d	16
Rectilinear . .	0·5	0·2d	32
,, . .	0·5	0·3d	24
,, . .	0·5	0·4d	16
,, . .	0·5	0·5d	8
,, . .	0·5	0·6d	0

The safe load on pillars should be obtained from the equation

$P = c[A + (m - 1)A_v]$ where

P = Total permissible *pressure* in lb.

c = working compressive stress lb. per sq. in. on the concrete of the hooped core.

W = actual load on pillar in lb.

A = effective area of the pillar in sq. in., i.e. the area bounded by the lateral reinforcement measured to the inside of the hooping.

A_v = Area of the *vertical* reinforcement in sq. in.

m = Modular ratio $= E_s/E_c = 15$.

The following limits of stress should be observed in pillars :—

(*a*) The stress on the steel must not exceed one-fourth of the ultimate tensile strength of the metal, or 0·5 of the elastic limit, or the value of mc.

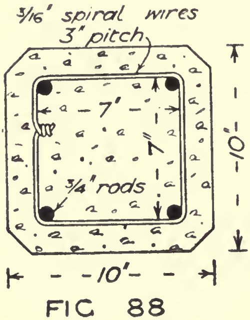
(*b*) Whatever the percentage of lateral reinforcement the working stress on the concrete of pillars must not exceed 0·5 C with rectilinear laterals, 0·58 C with independent circular hoops, 0·66 C with helical reinforcement.

Example of Calculation of Pillar Axially Loaded.—

A reinforced concrete pillar 10 in. square, 15 ft. high, is reinforced with four $\frac{3}{4}$ in. rods, bound together spirally at intervals of 3 in. with $\frac{3}{16}$ in. diameter wires, and carries an axial load of

20 tons, as in Fig. 88. Find the stresses on the steel and the concrete.

Ratio of length to least diameter $= \frac{15 \times 12}{10} = 18$, therefore the full stress may be allowed on the pillar. Total area of steel $4 \times {\cdot}75^2 \times {\cdot}7854 = 1{\cdot}76$ sq. in.



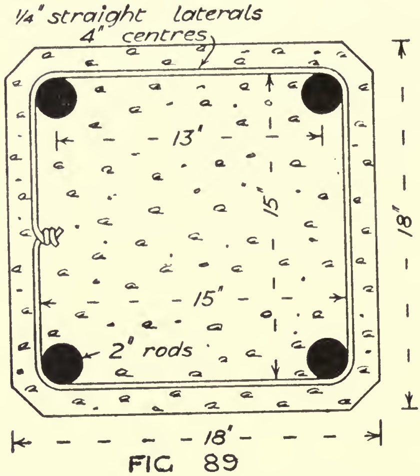


FIG. 88.—Cross-section of reinforced concrete pillar. FIG. 89.—Cross-section of reinforced concrete pillar from calculations.

Stress in concrete

$$c = \frac{W}{A + A_v(m-1)} = \frac{20 \times 2240}{49 + 1{\cdot}76(15-1)} = 608 \text{ lb. per sq. in.}$$

Maximum permissible stress in concrete

$$= kC(1 + fsr) = \tfrac{1}{4} \times 2400(1 + 1 \times 16 \times {\cdot}005) = 648 \text{ lb. per sq. in.}$$

Stress in steel,

$$t = \frac{mW}{A + A_v(m-1)} = \frac{15 \times 20 \times 2240}{49 + 1{\cdot}76(15-1)} = 9125 \text{ lb. per sq. in.}$$

the maximum permissible for pillars being 12,000 lb. per sq. in.

The maximum stress on an eccentrically loaded pillar will be given by the formula,

$$f = \frac{W}{A} \pm \frac{W'x}{Z}$$

where f = maximum stress in lb. per square inch at edge of section,

W = total load from all sources in lb.

A = equivalent area in square inches

$$= A + (m - 1)\,A_v$$

W' = eccentric load in lb.

x = eccentricity or distance in inches from neutral axis of pillar to centre of application.

Z = section modulus of pillar.

The section modulus of a rectangular reinforced concrete pillar

$$Z = \tfrac{1}{6}A_c h + \tfrac{1}{2}(m-1)A_v \frac{h_t^2}{h}$$

h is the whole diameter of the pillar at right angles to the neutral axis, h_t is the diameter from centre to centre of reinforcement.

The section modulus of a circular reinforced pillar with four bars,

$$Z = \tfrac{1}{8}A_c h + \tfrac{1}{2}(m-1)A_v \frac{h_t^2}{h}$$

The section modulus of a circular reinforced pillar with bars arranged in a circle,

$$Z = \tfrac{1}{8}A_c h + \tfrac{1}{4}(m-1)A_v \frac{h_t^2}{h}.$$

Example of Calculation of Eccentrically Loaded Pillar.—A rectangular reinforced concrete pillar 18 in. square, with four 2 in. rods, 13 in. centre to centre, 16 ft. high, with straight laterals $\tfrac{1}{4}$ in. diameter, 4 in. centres, has an axial load of 50 tons, and a load of 20 tons applied at the side, as in Fig. 89. Find the stresses on the two faces.

Area of steel $= 2^2 \times \cdot 7854 \times 4 = 12\cdot 5664$

Area of pillar $= 15 \times 15 = 225$

Equivalent area $= A + (m-1)A_v =$

$$225 + 14 \times 12\cdot 5664 = 400.$$

Section modulus $Z = \tfrac{1}{6}A_c h + \tfrac{1}{2}(m-1)A_v \dfrac{h_t^2}{h}$

$$= \tfrac{1}{6} \times 324 \times 18 + \tfrac{1}{2} \times 14 \times 12\cdot 5664 \times \frac{13^2}{18} = 1798.$$

The stresses on the two faces are,

$$f = \frac{W}{A} \pm \frac{W'x}{Z}$$

$$= \frac{(50+20)2240}{400} \pm \frac{20 \times 2240 \times 9}{1798} = 392 \pm 224$$

= 616 lb. per sq. in. compression under eccentric load

= 168 ,, ,, ,, on opposite face.

The allowable stress $= kC(1 + fsr)$

$= \frac{1}{4} \times 2240(1 + 0{\cdot}5 \times 24 \times {\cdot}003)$

$= 621$ lb. per sq. in.

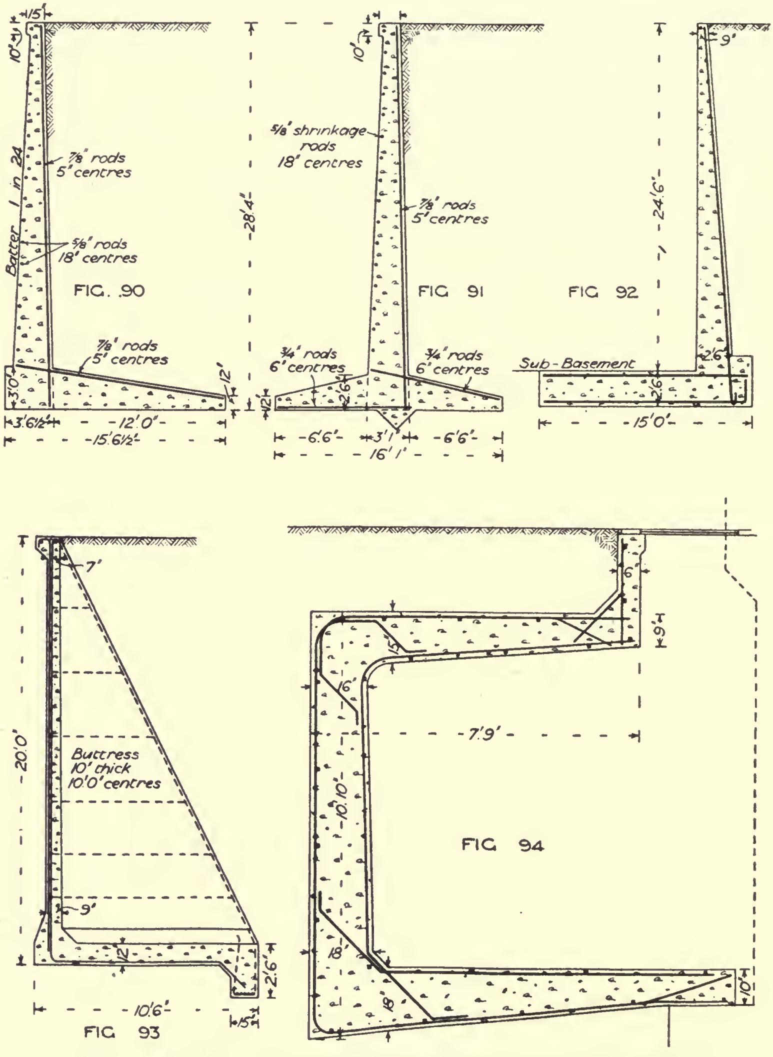
and the pillar is therefore safe.

CHAPTER V.

SPECIAL CONSTRUCTIONS.

Retaining Walls.—Retaining walls of reinforced concrete are built upon principles quite contrary to those of ordinary retaining walls. The latter depend for their stability upon their mass and weight, and the line of thrust, compounded of their weight acting through the centre of gravity and the thrust of the earth which they support acting at one-third of their height, must fall sufficiently within the face of the wall to leave the maximum stress of compression at the outer edge and tension at the inner edge within safe working limits. It is generally assumed that this line of thrust must fall within the middle third of the base, but this is not essential unless it be desired that there shall be a complete absence of tension. Reinforced concrete retaining walls rely chiefly upon their strength, and when they are properly designed they only require to be about as many inches in thickness as the old-fashioned walls require to be feet. There are only a few principal types but many modifications. Fig. 90 shows an L-shaped wall, where the foot is held down by the weight of the earth and the thrust is resisted by the upright cantilever. In Fig. 91 the wall is placed in the centre of the base, and in Fig. 92 the wall is placed at the inner edge of the base. In these two cases the wall has a tendency to lift, rotating on the outer edge of base. In Fig. 93 the same type is strengthened by counterforts every 10 ft. on the inside; there is also a cleat, or projection, on the underside to prevent the wall from being pushed out. In Fig. 94 the wall takes the form of two cantilevers, the upper one being loaded by the weight of footpath and the lower one by the reaction of the earth under foundation.

The weight and natural slope of various soils required in calculating retaining walls are given in the following table. They must be taken as approximate only, as no two writers



Figs. 90 to 94.—Sections of reinforced concrete retaining walls.

agree, and considerable variation will arise under different climatic conditions.

Soil.	lb. per cub. ft. (w).	Angle of Repose (θ).
Vegetable earth . .	90	30
Sandy loam . . .	100	34
Loamy clay . . .	110	36
Firm gravel . . .	120	40
Loose gravel . . .	110	36
Stiff clay	128	45
Wet clay	120	16

Fig. 95 shows a graphic diagram of the pressures on the vertical and horizontal portions of a retaining wall like Fig. 90. The bending moment at the foot of the wall per foot run is

$$B_e = \tfrac{1}{6} w h^3 \tan^2\left(45 - \frac{\theta}{2}\right).$$

The moment of resistance of cantilever (at foot of wall) as shown in last chapter is

$$R_t = tA_t\left(d - \frac{n}{3}\right), \text{ or } R_c = \frac{c}{2}bn\left(d - \frac{n}{3}\right),$$

whichever is the lesser. To balance the horizontal thrust the theoretical length of base is

$$l = \sqrt{\tfrac{1}{3}h^2 \tan^2\left(45 - \frac{\theta}{2}\right)}.$$

For stability the actual length of base should be not less than say

$$L = 1\tfrac{1}{2}l.$$

To find the pressure on the foundation an actual case must be taken, Fig. 96, say wall 20 ft. high, earth 90 lb. per cubic foot., natural slope 35°, then the horizontal thrust at $\frac{1}{3}$ the height is

$$T = \tfrac{1}{2} w h^2 \tan^2\left(45 - \frac{\theta}{2}\right)$$

$$= \tfrac{1}{2} \times 90 \times 20^2 \times \cdot 52^2$$

$$= 4867 \text{ lb.}$$

The theoretical length of base is

$$l \geqq \sqrt{\tfrac{1}{3}h^2 \tan^2\left(45 - \frac{\theta}{2}\right)}$$

that is, equal to or greater than,

$$\sqrt{\tfrac{1}{3} \times 20^2 \times \cdot 52^2} = 6 \text{ ft.}$$

but for stability $L = 1\frac{1}{2}l = 1\frac{1}{2} \times 6 = 9$ ft. Then, assuming the wall to be 9 in. thick (average) the weight will be

$$\cdot 75 \times 20 \times 150 = 2250 \text{ lb.}$$

The weight of earth on the base will be

$$whL = 90 \times 20 \times 9 = 16200 \text{ lb.}$$

The mean centre of gravity of loads will be

$$\frac{2250 \times 9 + 16200 \times \frac{9}{2}}{2250 + 16200} = 5\cdot 04 \text{ ft.}$$

from inner edge of base. The resultant of the thrust and loads will then cut the base at

$$\frac{\frac{1}{3} \times 20 \times 4867}{2250 + 16200} = 1\cdot 76 \text{ ft.}$$

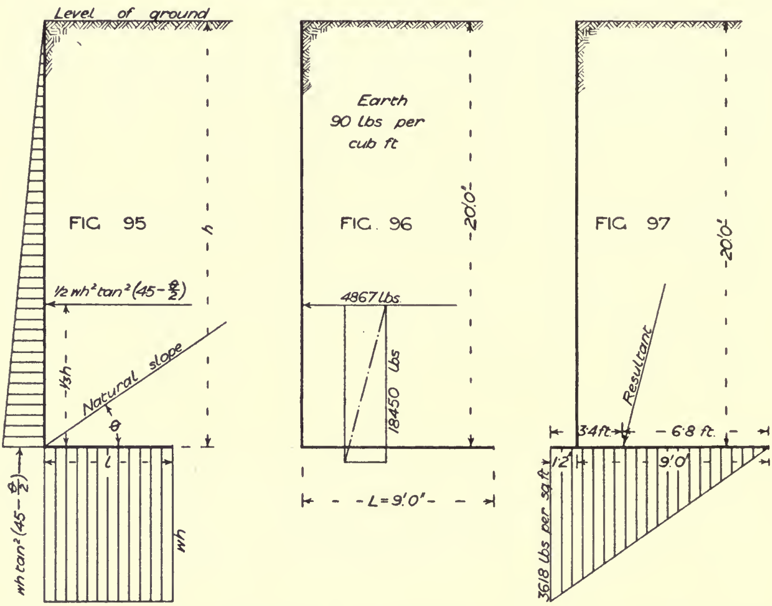


Fig. 95.—Diagram of horizontal and vertical pressures on reinforced concrete retaining wall like Fig. 90. Fig. 96.—Horizontal pressure against given wall. Fig. 97.—Vertical pressure on foundation of given wall.

beyond the centre of gravity line, or $5\cdot 04 + 1\cdot 76 = 6\cdot 8$ ft. from inner edge of base. As this is outside the middle third of base it is evident that there would be tension at the inside edge which is not permissible. The base must therefore be extended, the simplest and best method being to make the distance from the point where the resultant cuts the base to the front edge half the distance from resultant to inner edge, thus, $\frac{6\cdot8}{2} = 3\cdot4$, and $6\cdot8 + 3\cdot4 = 10\cdot2$ ft. total width of base, projecting

$$10\cdot2 - 9 = 1\cdot2 \text{ ft.}$$

in front of wall as in Fig. 97. The maximum pressure on foundation will then be $\frac{2250 + 16200}{10\cdot2} \times 2 = 3618$ lb. or 1·61 tons per square foot.

An example may now be taken for calculations like Fig. 93. In this case, instead of 1 ft. run, the whole length of one bay must be considered. Assume the same general dimensions and external conditions as before, and the counterforts 10 ft. centre to centre. The wall will not now be a cantilever but equivalent to a piece of floor slab with one free edge, and the counterfort will be a cantilever in the form of a tee beam.

Total pressure at $\frac{1}{3}$ height of wall

$$= \tfrac{1}{2}wh^2\mathrm{D} \tan^2\left(45 - \frac{\theta}{2}\right)$$

$$= \tfrac{1}{2} \times 90 \times 20^2 \times 10 \times \cdot52^2 = 48{,}670 \text{ lb.}$$

Weight of wall $= (\frac{7}{12} \times 20 \times 10)150 = 17{,}500$ lb.

Weight of counterfort $= (\frac{20}{2} \times 9 \times \frac{9}{12})150 = 10{,}125$ lb.

Weight of earth $= 90 \times 20 \times 9 \times 10 = 162{,}000$ lb.

Mean c.g. of loads

$$= \frac{(17500 \times 9) + (10125 \times \frac{2}{3} \times 9) + (162000 \times \frac{9}{2})}{17500 + 10125 + 162000}$$

$= 5$ ft. from inner edge of base.

The resultant of thrust and loads will then cut the base at $\frac{\frac{1}{3} \times 20 \times 48670}{17500 + 10125 + 162000} = 1\cdot71$ ft. beyond the c.g. line or $5 + 1\cdot7 = 6\cdot7$ ft. from inner edge of base as in Fig. 98. The base should therefore be extended to $6\cdot7 + \frac{6\cdot7}{2} = 10$ ft. The maximum pressure on foundation will then be

$$\frac{17500 + 10125 + 162000}{10 \times 10} \times 2 = 3792\cdot5 \text{ lb.}$$

or 1·69 tons per sq. ft., as in Fig. 99.

Bending moment on cantilever tee beam

$$= 48670 \times \tfrac{20}{3} \times 12 = 3{,}893{,}600 \text{ lb.-in.}$$

Assuming three 1 in. rods as reinforcement

$$p = \frac{100 \times 2\cdot3562}{84 \times 7} = 0\cdot4$$

$$n = 105 \times \frac{0\cdot5 \times \frac{7}{105} + 0\cdot15 \times 0\cdot4}{1 + 0\cdot15 \times 0\cdot4} = 9\cdot45 \text{ in.}$$

$$z = \tfrac{7}{3} \times \frac{3 \times 9\cdot45 - 2 \times 7}{2 \times 9\cdot45 - 7} = 2\cdot81 \text{ in.}$$

$$c = \frac{2 \times 3{,}893{,}600 \times 9\cdot45}{84 \times 7(2 \times 9\cdot45 - 7)(105 - 2\cdot81)} = 103 \text{ lb. per sq. in.}$$

$$t = \frac{3{,}893{,}600}{2\cdot3562(105 - 2\cdot81)} = 16{,}170 \text{ lb. per sq. in.}$$

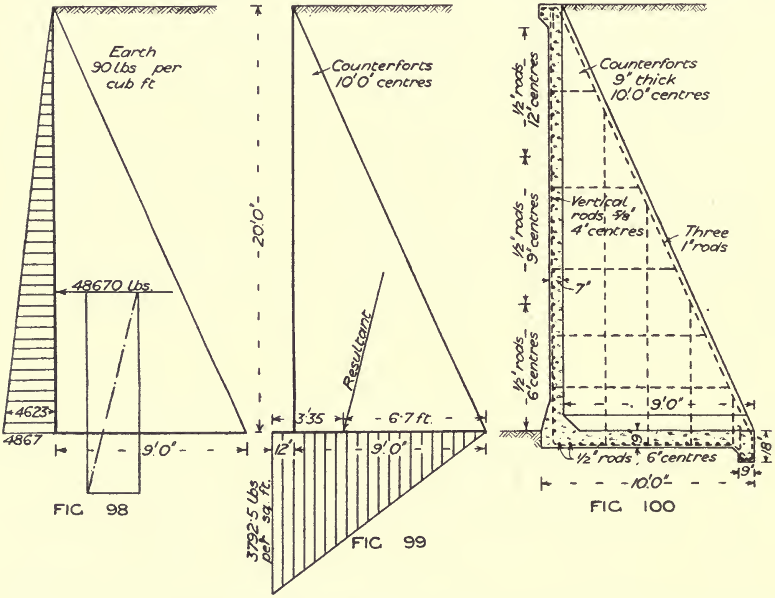


FIG. 98.—Horizontal pressures against given retaining wall like Fig. 93. FIG. 99.—Vertical pressures on foundation of given retaining wall like Fig. 93. FIG. 100.—Completed design of retaining wall with reinforcement.

It will be sufficiently near in most cases to take the bending moments on the base between counterforts as $\frac{wl^2}{16}$, w being the weight of earth per foot super on base and l the width of base.

For the vertical wall W the total load will be the thrust and l will be the height, and the bending moment at the bottom will be $\frac{Wl}{24}$. If the top of wall is stiffened sufficiently to form a girder carrying full stress it will be designed as described in the section on bins.

The maximum bending moment on base

$$\frac{wl^2}{16} = \frac{(20 \times 90)(9 \times 12)^2}{12 \times 16} = 109{,}350 \text{ lb.-in.}$$

The effective depth of base slab $= 8$ in. and the reinforcement per foot wide $= 3 \times 0{\cdot}3 = 0{\cdot}9$ sq. in., then $r = \frac{0{\cdot}9}{12 \times 8} = {\cdot}01$, $m = 15$.

$$k = \sqrt{({\cdot}01^2 \times 15^2 + 2 \times {\cdot}01 \times 15)} - {\cdot}01 \times 15 = {\cdot}42$$

$$1 - \tfrac{1}{3}k = 1 - \frac{{\cdot}42}{3} = 0{\cdot}86$$

$$t = \frac{109350}{{\cdot}01 \times 12 \times 8^2({\cdot}86)} = 16{,}550 \text{ lb. per sq. in.}$$

$$c = \frac{2 \times 109350}{{\cdot}42 \times 12 \times 8^2 \times ({\cdot}86)} = 777 \text{ lb. per sq. in.}$$

but this is making no allowance for the projection on inner edge of base.

The maximum bending moment on vertical wall

$$= \frac{Wl}{24} = \frac{48670 \times (20 \times 12)}{10 \times 24} = 48670 \text{ lb.-in.}$$

The effective depth (thickness) at bottom of wall $= 6$ in. and the reinforcement per foot, width $= 3 \times 0{\cdot}3 = 0{\cdot}9$ sq. in. then

$$r = \frac{0{\cdot}9}{12 \times 6} = {\cdot}0125, \quad m = 15$$

$$k = \sqrt{({\cdot}0125^2 \times 15^2 + 2 \times {\cdot}0125 \times 15)} - {\cdot}0125 \times 15$$
$$= 0{\cdot}453$$

$$1 - \tfrac{1}{3}k = 1 - \frac{0{\cdot}453}{3} = 0{\cdot}849$$

$$t = \frac{48670}{{\cdot}0125 \times 12 \times 6^2({\cdot}849)} = 10{,}620 \text{ lb. per sq. in.}$$

$$c = \frac{2 \times 48670}{{\cdot}453 \times 12 \times 6^2({\cdot}849)} = 586 \text{ lb. per sq. in.}$$

The wall slab must also be calculated in the other direction in order to obtain the horizontal reinforcement. The maximum bending moment will be $B_e = \frac{Wl}{20}$ where W = earth pressure on wall varying from maximum at bottom to nil at top of wall, and l = span or centre distance of the counterforts. In the present case taking the bottom foot of wall $w = \frac{4623 + 4867}{2} = 4745$ lb. and $B_e = \frac{4745 \times 10 \times 12}{20} = 28470$ lb.-in. The effective depth of wall = 6 in. and the reinforcement may be ½ in. rods, 6 in. centres or $2 \times 0{\cdot}2 = 0{\cdot}4$ sq. in. in the bottom foot of wall.

$$\text{Then } r = \frac{0{\cdot}4}{12 \times 6} = {\cdot}006,\ m = 15$$

$$k = \sqrt{({\cdot}006^2 \times 15^2 + 2 \times {\cdot}006 \times 15)} - {\cdot}006 \times 15 = 0{\cdot}34.$$

$$1 - \tfrac{1}{3}k = 1 = \frac{{\cdot}34}{3} = 0{\cdot}89$$

$$t = \frac{28470}{{\cdot}006 \times 12 \times 6^2({\cdot}89)} = 12{,}340 \text{ lb. per sq. in.}$$

$$c = \frac{2 \times 28470}{{\cdot}34 \times 12 \times 6^2({\cdot}89)} = 436 \text{ lb. per sq. in.}$$

Similarly the spacing may be found at any point in the wall.

The complete design may now be drawn out as in Fig. 100.

Bins.—Bins, bunkers, hoppers, pockets, silos or cells, as they are variously called, are modified retaining walls, they are boxes, circular or rectangular, generally of considerable height in proportion to their diameter, built to contain grain, coal, cement, ore, or similar material.

When the width is not less that half the depth the pressure at any point of the depth will be found exactly in the same way as against a retaining wall; with less width in proportion to the depth the friction against the vertical sides will become appreciable and require to be allowed for. To enable the necessary calculations to be made, the weight of the material per cubic foot and the natural slope or angle of repose, are required.

Material.	(w) lb. per cub. ft.	(θ) Angle of Repose.
Ore	100-150	35-45
Slag (small)	100-112	45
Coal (house)	50-55	36-40
,, (gas)	56-58	36-40
,, (Welsh)	70-75	36-40
Coke	25-35	35-45
Shingle	88	39
Sand (river)	110	30
,, (pit)	110	30
Cement (Portland)	85-95	25-35
Wheat	50	25
Barley	39	26
Oats	28	24
Malt	33	22

Fig. 101 is the vertical-section of a coal bunker to carry heavy coal say 75 lb. per cubic foot with provision for surcharge, which should always be made. It is assumed that the length in the other direction is such that the centre is unaffected by the tying in of the ends. The pressure per square foot at the top will be nil, increasing downwards until at the bottom of the vertical side it is equal to $wh \cos^2 \theta$. Upon the sloping sides of the bottom, lying at an angle of 45°, the pressure per square foot at the upper edge will be $wh \cos \theta$ and at the lower edge $w\text{H} \cos \theta$. The pressure per square foot on the flat bottom will be $w\text{H}$.

If the top edge of bunker is stiffened by a flange or girder so that the pressures are transmitted to the ends, then one-third of the total thrust will be taken by the top edge and two-thirds by the bottom edge, the upright portion being subject to a maximum bending moment of $\text{B} = \frac{1}{9} wh^3 \cos^2 \theta$ occurring at a height of $\frac{1}{3} h$ from the bottom.

Mr. E. F. Etchells has given a simple series of formulæ for coal hoppers as follows :—

Horizontal thrust at $\frac{1}{3}$ height, in tons per foot run, coal level with the top $= \dfrac{\text{Height in feet}^2}{240}$.

If surcharged to the full angle, thrust $= \dfrac{\text{Height in feet}^2}{100}$.

If the sides are tied in, horizontal thrust at top $\frac{1}{3}$ of above, and at bottom $\frac{2}{3}$ of above.

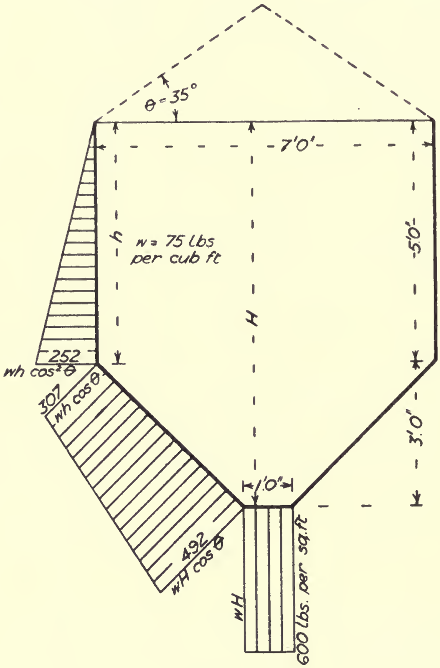


FIG 101

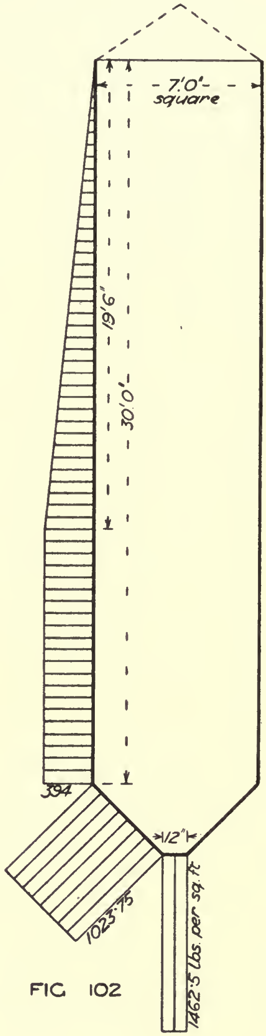


FIG. 101.—Vertical section of coal bunker with diagrams of pressure. FIG. 102.—Diagram of pressures on high bin or bunker of same horizontal measurements as Fig. 101.

Maximum bending moment on upright = Thrust × $\frac{1}{8}$ Height.
When the width of bin is less than half the depth the greatest

pressure per square foot against the side will be given by the formula

$$p = \frac{Aw}{P \tan \phi}$$

where p = maximum pressure against side in lb. per square foot.

w = weight of material in lb. per cubic foot.

P = perimeter of bin in feet.

A = area of bin in square feet.

$\tan \phi$ = coefficient of friction of material against wall
= say from $\frac{1}{4}$ to $\frac{1}{3}$

and the depth (d) in feet from the surface to point of maximum pressure is

$$d = \frac{A}{P \tan \phi \tan^2 \left(45 - \frac{\theta}{2}\right)}$$

the pressure at the top will be zero increasing gradually to maximum at this depth, then remaining constant to the bottom. On the horizontal base the maximum pressure (q) per square foot will be

$$q = wd.$$

If the bottom slopes at angle θ the pressure per square foot on it will be $q \sin \theta$.

As an example, a square bin may be taken of the same horizontal measurement as Fig. 101 but a height (h) of 30 ft. with coal as before.

$$\text{Then } p = \frac{Aw}{P \tan \phi} = \frac{7 \times 7 \times 75}{7 \times 4 \times \frac{1}{3}} = 394 \text{ lb. per sq. ft.}$$

$$d = \frac{A}{P \tan \phi \tan^2 \left(45 - \frac{\theta}{2}\right)^2} = \frac{7 \times 7}{7 \times 4 \times \frac{1}{3} \times \cdot 52^2} = 19 \cdot 5 \text{ ft.}$$

$$q = wd = 75 \times 19 \cdot 5 = 1462 \cdot 5 \text{ lb. per sq. ft.}$$

On the sloping bottom $p_1 = q \cos 45° = 1462 \cdot 5 \times \cdot 7 = 1023 \cdot 75$ lb. per square foot. On the horizontal bottom $p_2 = q = 1462 \cdot 5$ lb. per square foot. These pressures are shown graphically on Fig. 102.

Arches.—Arches in reinforced concrete are subject to the same principle of stress as when built of other material,[1] but as their construction admits of a greater intensity of stress in tension this point must receive special attention. If the abutments of an arch are not rigid the arch becomes a beam with the minimum depth where the greatest bending moment occurs, and it is therefore essential that a reinforced concrete arch should have rigid abutments although the material can be designed to resist tension. The curve of thrust must be drawn upon the elevation of the arch, as with other materials, and the maximum stresses in tension and compression must be provided for in the design. Fig. 103 shows an arch formed of two circular arcs,

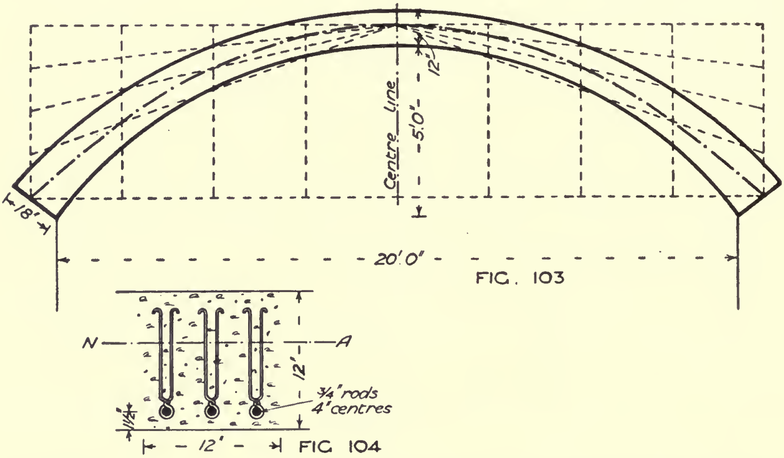


Fig. 103.—Curve of thrust on arch. Fig. 104.—Section at crown of arch showing reinforcement.

under a load distributed uniformly over the horizontal span, and Fig. 104 a section at the crown of arch. Under this condition we know that the curve of thrust will be a parabola and it may therefore be drawn in without any preliminaries; but there are various possible lines of thrust according to the points taken for them to pass through at the crown and skewbacks. Nature chooses that line of thrust which will, on the whole, give the

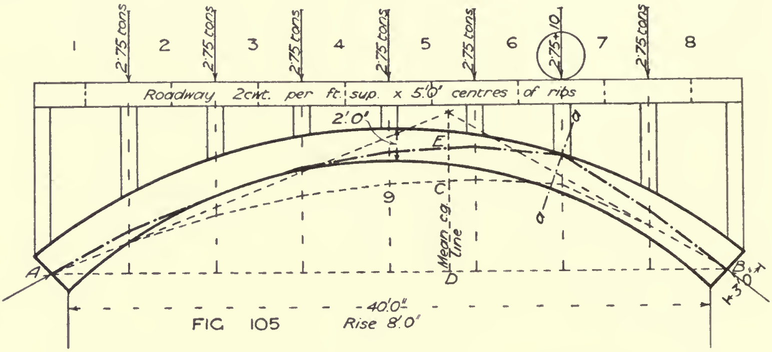
[1] See Paper on "The Stability of Arches," by Henry Adams. "Trans. C. & M.E.S.," 4 Feb. 1909.

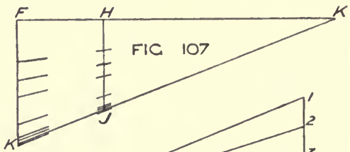
minimum stresses, and although we may endeavour to find it by trial we cannot always succeed, particularly when moving loads have to be taken into account. In the present case we may assume it to pass through the neutral axis at the crown and at the skewbacks. The thrust at the crown will be $\frac{wl^2}{8r}$ where w = load per foot run, l = span of line of thrust, r = rise of line of thrust, equivalent to the $\frac{wl^2}{8d}$ of a flanged beam.

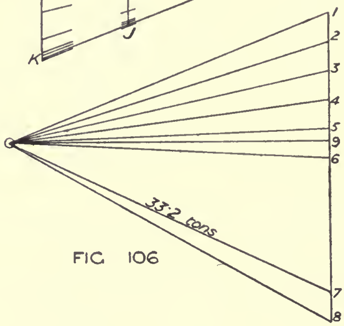
The curve of thrust should pass through the neutral axis of the rib and the maximum thrust on the concrete will then be $\frac{W}{A}$ where W = total thrust at each point of the curve and A = the area of concrete above the reinforcement + 14 times the area of the steel.

In nearly all practical cases a reinforced concrete arch has to be designed for a rolling load in addition to the dead load of the structure. It is usual also to provide for an external distributed load of say $1\frac{1}{2}$ cwt. per foot super. A rolling load, say a traction engine, may bring a load of 10 tons on one point, and the position where it will cause a maximum distortion of the line of thrust may be taken as about one-fourth of the span, but both these loads will not occur at the same time. The loads are often transmitted from the surface of the roadway to the arch ring by means of vertical pillars, and the arch ring will consist of a thin shell with deep ribs at intervals. Consider one of the arch ribs as in Fig. 105 loaded as shown. Then by calculation the sum of the loads at $A = \frac{19{\cdot}25}{2} + 10 \times \frac{10{\cdot}25}{42} = 12{\cdot}065$ tons and at B $\frac{19{\cdot}25}{2} + 10 \times \frac{31{\cdot}75}{42} = 17{\cdot}185$ tons. Set down the load line 1 to 8 in Fig. 106, and mark off the calculated reactions, giving point 9. Select any point 0 horizontally from 9 and draw vectors to the points on the load line, and parallel with these vectors, across their respective spaces, draw the funicular polygon ACB in Fig. 105, producing the lines across spaces 1 and 8 to intersect in a point on the mean centre of gravity line. Bisect the depth of arch ring on this line at E, then in Fig. 107

set up FG = DE and at any convenient distance from it set up HJ = DC. Produce GJ and FH to meet in K. Then mark off on HJ the heights of the funicular polygon ACB, join to K and produce to cut FG, thus obtaining magnified ordinates to the curve of thrust AEB which passes through centre of depth of arch on the mean centre of gravity line. The maximum stresses will







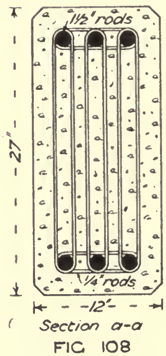


FIG. 105.—Reinforced concrete arch with ribs and pillow supporting roadway for rolling load. FIG. 106.—Vector diagram for Fig. 105. FIG. 107.—Diagram for raising line of thrust to required height. FIG. 108.—Section of arch rib showing reinforcement.

occur across space 7 where the thrust is 33·2 tons with an eccentricity of 1 ft. 1½ in., but the thrust may be taken as 35 tons, as the weight of rib has not been allowed for in the previous working. The arch may be treated as a column with a section as shown in Fig. 108.

Then area of steel $= 1\frac{1}{2}^2 \times \cdot 7854 \times 6 = 10\cdot 6$ sq. in.
Area of section $= 22 \times 10 = 220$ sq. in.
Equivalent area (A) $= 220 + 14 \times 10\cdot 6 = 368$ sq. in.
Section modulus (Z) $= (\frac{1}{6} \times 27 \times 12 \times 27) + (\frac{1}{2} \times 14 \times 10\cdot 6 \times \frac{21^2}{27})$
$= 2770.$

The stresses on the two faces are

$$f = \frac{W}{A} \pm \frac{W'x}{Z}$$
$$= \frac{(35 \times 2240)}{368} \pm \frac{35 \times 2240 \times 13\cdot 5}{2770}$$
$$= 213 \pm 382$$

$= 595$ lb. per sq. in. compression on the loaded side and 169 lb. per sq. in. tension on opposite face, but as tension is produced it is advisable to work by the formula for doubly reinforced beams, as the formula $\frac{W}{A} \pm \frac{M}{Z}$ does not appear to be suitable for reinforced concrete when tension occurs. The equivalent area of the section $= (12 \times 27) + 14(5\cdot 3 + 5\cdot 3) = 472\cdot 4$ sq. in. Therefore thrust $= \frac{35 \times 2240}{472\cdot 4} = 166$ lb. per sq. in. The maximum allowable compression on the concrete to resist the bending moment $= 600 - 166 = 434$ lb. per sq. in., and the maximum allowable tension on the steel $= 16000 + 15 \times 166 = 18{,}490$ lb. per sq. in. The bending moment $B = 35 \times 2240 \times 13\cdot 5 = 1{,}058{,}400$ lb.-in. Then by the formulæ for doubly reinforced beams

$$n = \frac{15}{12}(5\cdot 3 + 5\cdot 3)\left[\sqrt{1 + \frac{2 \times 12(3 \times 5\cdot 3 + 24 \times 5\cdot 3)}{15(5\cdot 3 + 5\cdot 3)^2}} - 1\right]$$
$$= 10 \text{ in.}$$
$$c = \frac{10 \times 1058400}{\frac{1}{3} \times 10^3 \times 12 + 15\{5\cdot 3(10-3)^2 + 5\cdot 3(24-10)^2\}}$$
$$= 450 \text{ lb. per sq. in.}$$
$$t = 450 \times 15\,\frac{24-10}{10} = 9450 \text{ lb. per sq. in.}$$

$c_s = 450 \times 15\,\frac{10-3}{10} = 4725$ lb. per sq. in., so that the design will be suitable.

Chimney Shafts.—Chimneys in reinforced concrete were at first made with so little taper as to be practically in the form of cylindrical pipes and were most unsightly. There have been some more recent examples with a considerable amount of taper, to their manifest advantage from an æsthetic point of view. There are two points connected with their design in which they differ from other structures in reinforced concrete. The first has to do with the question of wind pressure and difficulties arise first from the great height making it important to determine accurately the normal pressure to be allowed for and next from the form making the effective pressure uncertain. Those who have studied the subject of wind pressure are aware that within certain limits it varies as the square of the velocity, so that until the wind reaches the velocity of a gale the pressures are only trifling. The following table shows the various descriptions of wind with the velocities and corresponding pressures against a plane surface perpendicular to its direction.

see p. 114

Description.	Velocity in Miles per hour.	Approximate Corresponding Pressure lb. per sq. ft.
Barely perceptible wind . .	$2\frac{1}{2}$	$\frac{1}{32}$
Light breeze	5	$\frac{1}{8}$
Pleasant breeze	$7\frac{1}{2}$	$\frac{1}{4}$
Good breeze	10	$\frac{1}{2}$
Strong breeze	15	$1\frac{1}{8}$
High wind	20	2
Half gale	30	$4\frac{1}{2}$
Strong gale	40	8
Whole gale	50	$12\frac{1}{2}$
Great storm	60	18
Hurricane	80	32
Violent hurricane . . .	100	50

In the discussion on a paper upon "The Stability of Chimney Shafts" read before the Society of Engineers in 1887, the writer called attention to the variation of wind pressure at different heights caused by the resistance offered to its motion by buildings, etc., upon the ground. The experiments of Sir B. Baker also showed that the pressure of the wind at one time was not equally diffused over a large area, and that it might be as much as 40 lb. per square foot over a small portion of a structure while at the same time the average pressure over the whole did not exceed 24 lb. per square foot. These two considerations show that a working formula for wind pressure should combine the elements of width and height. An empirical formula designed by the writer for this purpose is

$$\log p = 1{\cdot}125 + 0{\cdot}32 \log h - 0{\cdot}12 \log w$$

where p = ultimate wind pressure in lb. per square foot necessary to be allowed for against a plane surface normal to the wind.

h = height of centre of gravity of surface considered above ground level in feet.

w = width in feet of part to be taken as one surface.

The following table shows the figures given by this formula for various widths and heights; Fig. 109 shows the curves of pressure given by the formula.

Height in ft.	Width in ft.						
	5	10	20	50	100	200	500
150	54·6	50·3	46·3	41·4	38·1	35·1	31·4
100	48·0	44·2	40·7	36·4	33·5	30·8	27·6
50	38·4	35·4	32·5	29·1	26·8	24·7	22·1
20	28·7	26·4	24·3	21·7	20·0	18·4	16·5
10	23·0	21·1	19·5	17·4	16·0	14·8	13·2
2	18·4	17·0	15·6	13·9	12·8	11·8	10·6

MULTIPLIERS FOR ANGLE.

0	10	20	30	40	50	60	70	80	90
sin	·174	·342	·500	·643	·766	·866	·940	·985	1
$\sin^2$	·0303	·117	·250	·413	·587	·750	·884	·970	1

When the exposed surface is not perpendicular to the direction of the wind, allowance must be made for what may be termed "slipping off," the effective pressure being reduced by the multipliers given in the above table according to the angle. The *sin* is used for obtaining the normal pressure on an inclined surface and the sin^2 for obtaining the effective pressure in the same direction as the wind. The additional curves marked Rogers Field and Thos. Stevenson are based upon formulæ by those engineers which take account of variation in height of object but not width. They appear to be about a mean of those given by preceding formula.

Mr. E. Fiander Etchells of the London County Council Architect's Department, has recently given a formula for

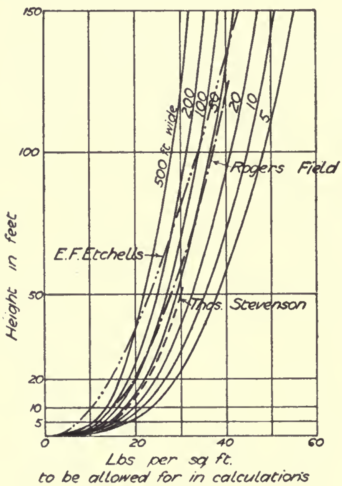


FIG 109

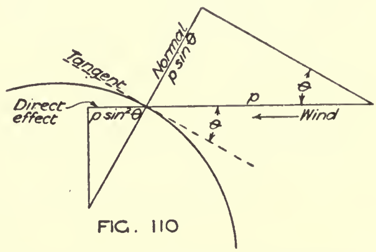


FIG. 110

FIG. 109.—Diagram showing variation of wind pressure according to height and width. FIG. 110.—Effective pressure on circular chimney.

variation of wind pressure to be allowed according to the width and height of the object, as follows:—

$$p = 3(\sqrt{h}) + 2\frac{h}{b}$$

where p = equivalent uniform wind pressure acting over whole surface exposed, in lb. per square foot.

h = height of top of structure above level of ground in feet.

b = breadth of part exposed to wind pressure in feet.

The curve produced by this formula for a width of 50 feet has been plotted upon Fig. 109 for comparison with the others.

A circular chimney stack will not come under the last table owing to the surface being curved and therefore presenting all the angles at one time. The multiplier or coefficient for a cylindrical surface is variously given by different authors as follows:—

Rankine	= ·5	Prof. Hutton	= ·66
Wilson	= ·55	Gaudard	= ·666
Borda	= ·57	Bressé	= ·78
Sir B. Baker	= ·57	Adams	= ·7854

The principle involved in obtaining this latter constant is shown in Fig. 110. Then the average pressure on the windward side will be the pressure of the wind per square foot on a normal plane multiplied by the summation of the squares of the sines of all the angles from 0° to 90°.

or $\Sigma\ (p \sin^2 0^\circ\ .\ .\ .\ 90^\circ) = {\cdot}5p$

or total result against the diametrical plane $= \frac{\pi}{2} \times {\cdot}5p = {\cdot}7854p$.

It should be noted that the square of the mean of several figures is not the same as the mean of the squares: for example, the square of the mean of 2, 4 and 6 is 16; while the mean of the squares of 2, 4 and 6 is $18\frac{2}{3}$. In the present case the mean of the sines is 0·627, the square of the mean 0·3931 and the mean of the square 0·5.

The opposite extreme of coefficient for wind pressure on circular chimneys appears to be that given in Lamb's "Hydrodynamics," p. 87, where the resistance is given as zero "in the absence of friction and under the critical velocity at which discontinuity takes place".

As a reinforced concrete chimney shaft has little weight it is necessary to tie it well into the foundation by hooking the vertical rods under the horizontal rods of the base, which should be well extended on all sides so that it will not overturn.

The height of the centre of pressure of the exposed surface of a circular tapered chimney shaft will be $h = \frac{H(D + 2d)}{3(D + d)}$, where

h = height of centre of pressure of wind, H = total height, D = outside diameter of base, d = outside diameter of top all in feet.

When the allowable stresses are 600 lb. per sq. in. on the concrete in compression and 16,000 lb. per sq. in. on the steel in tension the thickness of chimney may be found by the formula $\frac{0{\cdot}24\text{M}}{\text{C}r^2}+\frac{0{\cdot}4\text{W}}{\text{C}r}$, and the total area of steel $\frac{{\cdot}065\text{M}}{\text{C}r}-\frac{{\cdot}055\text{W}}{\text{C}}$ where W = the weight of chimney above the section under consideration in lb.

M = the bending moment due to wind about this section.

C = the compressive stress in the concrete in lb. per sq. in.

r = the inside radius of the shell in inches.

As the bending moment reduces in proportion to the distance from the base the reinforcement would be reduced as the chimney rises. Calculations by the above formula would be made at say 20 ft. intervals to ascertain the proper allowance.

An example may be taken for calculation, say chimney stack 120 ft. high, 6 ft. outside diameter at the top, and 8 ft. 6 in. outside diameter at the base. Foundation base 20 ft. square, bottom of foundation 12 ft. below surface of ground.

$$\text{Then } h=\frac{120(8{\cdot}5+2\times 6)}{3(8{\cdot}5+6)}=56{\cdot}55 \text{ ft.}$$

$$\text{and } \log p = 1{\cdot}125+0{\cdot}32\log 56{\cdot}55-0{\cdot}12\log\frac{8{\cdot}5+6}{2}$$

$$=1{\cdot}125+0{\cdot}561-0{\cdot}103=1{\cdot}583$$

$$\text{or } p=38{\cdot}28 \text{ lb. per sq. ft.}$$

or the total pressure on windward side of chimney

$$=120\times\frac{8{\cdot}5+6}{2}\times 0{\cdot}7854\times 38{\cdot}28$$

$$=26{,}160 \text{ lb.}$$

$$\text{and M}=26160\times 56{\cdot}55\times 12$$

$$=17750000 \text{ lb.-in.}$$

Weight of chimney with average thickness of 6 in.

$$=1586{\cdot}5\times 150=237975 \text{ lb.}$$

$$\text{Then thickness of chimney}=0{\cdot}24\times\frac{17750000}{600\times 45^2}+0{\cdot}4\times\frac{237975}{600\times 45}$$

$$=3{\cdot}51+3{\cdot}52=7{\cdot}03 \text{ in.}$$

or say 7 in. thickness at base.

$$\text{Area of steel} = \cdot 065 \times \frac{17750000}{600 \times 45} - \cdot 055 \times \frac{237975}{600}$$

$$= 42 \cdot 73 - 21 \cdot 81 = 20 \cdot 92 \text{ sq. in.}$$

or say $\frac{3}{4}$ in. rods at 6 in. centres.

The strength of this construction may be calculated as follows.

The equivalent area of section shown in Fig. 111 =

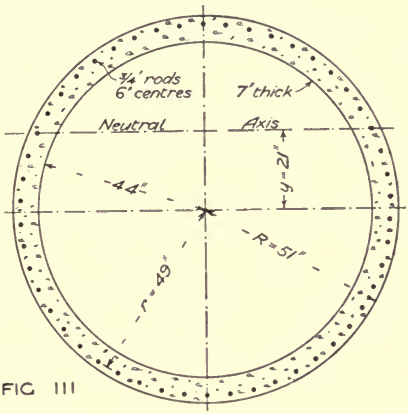


Fig. 111.—Cross-section of circular reinforced concrete chimney showing neutral axis.

$$\frac{3 \cdot 1416}{4} (102^2 - 88^2) + 14 \times 22 = 2397 \text{ sq. in.}$$

$$\text{Therefore thrust } (c) = \frac{237975}{2397} = 99 \text{ lb. per sq. in.}$$

The maximum allowable compression on the concrete to resist the bending moment (C) = 600 − 99 = 501 lb. per sq. in., and the maximum allowable tension on the steel

$$= 16000 + 15 \times 99 = 17485 \text{ lb. per sq. in.}$$

The distance from centre of section to neutral axis by the formula in Marsh and Dunn's manual of Reinforced Concrete,

$$y = \frac{R - \dfrac{c_b}{1067 + c} r}{1 + \dfrac{c_b}{1067 + c}} = \frac{51 - \dfrac{501}{1067 + 99} \times 49}{1 + \dfrac{501}{1067 + 99}} = 21 \text{ in.}$$

and $n = 51 - 21 = 30$ in.

$d = 102 - 2 = 100$ in.

$b = 0{\cdot}59,\ d = 0{\cdot}59 \times 100 = 59$ in.

$x = 2$ in.

$A = 18 \times {\cdot}4417 = 7{\cdot}9$

$A_t = 32 \times {\cdot}4417 = 14{\cdot}1$

Then by the formula for doubly reinforced beams

$$c = \frac{30 \times 17750000}{\frac{1}{3} \times 30^3 \times 59 + 15\{7{\cdot}9(30-2)^2 + 14{\cdot}1(100-30)^2\}}$$

$$= 321 \text{ lb. per sq. in.}$$

$$t = 321 \times 15\ \frac{100-30}{30} = 11235 \text{ lb. per sq. in.}$$

$$c_s = 321 \times 15\ \frac{30-2}{30} = 4494 \text{ lb. per sq. in.}$$

and the design therefore appears to be suitable. The upper stages of the shaft would be calculated in a similar manner.

It would appear that the strength of the chimney shaft might have been calculated by taking it as a column under direct load and bending moment, and deducting the value of the central core. The method would be as follows:—

Equivalent area of concrete and steel to edge of reinforcement $= \frac{3{\cdot}1416}{4}(102^2 - 88^2) + 14 \times 22 = 2397$ sq. in.

Total area of base $= \frac{3{\cdot}1416}{4} \times 102^2 = 8171$ sq. in.

Section modulus

$$= \left\{(\tfrac{1}{8} \times 8171 \times 102) + \tfrac{1}{4}(15-1)23 \times \frac{98^2}{102}\right\} - \frac{3{\cdot}1416}{32} \times 88^3$$

$$= 104180 + 7580 - 66903$$

$$= 44857 \text{ in. units.}$$

$$\frac{W}{A} \pm \frac{M}{Z} = \frac{237975}{2397} \pm \frac{17750000}{44857} = 99 \pm 396$$

$= 495$ lb. per sq. in. compression

297 ,, ,, tension.

The tension given is that upon the concrete and the concrete equivalent of the steel. It would represent a tension on the steel of $297 \times 15 = 4455$ lb. per square inch, but this differs so considerably from the previous calculations that the method does not appear to be applicable.

CHAPTER VI.

EFFECTS OF EXCESSIVE HEAT ON CONCRETE AND REINFORCED CONCRETE. TALL CHIMNEY CONSTRUCTION. EFFECTS OF FROST ON CONCRETE.

Effects of Excessive Heat on Concrete and Reinforced Concrete. —The subject of the fire resistance of reinforced concrete has from time to time been fully discussed, and the general conclusion arrived at is that the material is an excellent fire resistant. This was brought into evidence most forcibly during the great San Francisco fire, for while there were in that city on the Pacific seaboard no buildings which were entirely of reinforced concrete, there were many in which the floors and beams were of that material, and these stood both the earthquake shocks and fire admirably.

At the International Fire Service Congress at Milan in 1906 the fire resistance of reinforced concrete was fully discussed, and the following important resolutions were passed :—

"That the Congress considers that no reinforced concrete construction should be permissible in buildings intended to be fire resisting, unless the aggregate be most carefully selected and applied in such a manner as to give substantial protection to all metal parts."

"That it is advisable where reinforced concrete is intended to be fire resisting, that every portion of the metal rods or bars contained therein be covered by not less than 2 in. of concrete, the aggregate of which must be able to pass through a sieve having a mesh of no more than 1 in. in diameter, and that Portland cement of great fineness only be used."

"That where feasible all external angles should be rounded."

"That any angle-iron needed for mechanical protection should be held in position independently of the concrete."

These resolutions are interesting coming as they do from a conference of men who are chiefly interested in the protection of life and property, and including the chiefs of the Fire Brigades of all the principal cities of Europe.

The British Fire Prevention Committee have carried out a series of important tests with reinforced concrete floors, beams, etc., with most satisfactory results, it being found that no appreciable effect occurred by the combined application of fire, in some cases up to 2000° F., followed by water for five minutes from a steam fire engine. Prof. Ira Woolson, M.E., in a paper read in 1906 before the American Society for Testing Materials, describes a series of interesting tests which he made with a view to finding out the thermal conductivity of concrete, and the effect of heat upon its strength and elastic properties.

Some points from his paper are as follows:—

Composition of Specimens.—Prof. Woolson's test specimens were composed of a 1 : 2 : 4 mixture of Portland cement, sharp sand, and ¾ in. clean broken stone, both trap rock and limestone being employed.

In addition to these, other specimens were made in the same proportions, using clean ½ in. quartz gravel instead of the broken stone, and, in another set, clean boiler cinders.

Slow Setting Cement.—The cement used was a very slow-setting cement, taking seven hours forty minutes for initial set, and fourteen hours ten minutes for hard set.

Tensile Strength of Cement Used.—

Tensile strength for neat cement	at	7	days	= 710 lb.
,, ,, 3 to 1 concrete	,,	7	,,	= 160 ,,
,, ,, ,,	,,	28	,,	= 286 ,,

The concrete was mixed moderately wet.

Age of Specimens.—The specimens varied in age from two months to two years.

The Two Months Blocks.—These were left in moulds thirty-six hours, then submerged in water for seven days, then allowed to stand in air for seven weeks, being occasionally sprinkled.

1500° F. Heat Test for Two to Five Hours.—The heat test was as follows:—

Three and five hours were chosen for the times of heating the large specimens, and two and three hours for the 4 in. cubes, and the temperature in the furnace in which these specimens were placed was raised to 1500° F. (average); the furnace was raised to this temperature (after the blocks had been inserted) in forty to sixty minutes. After heating, the specimens were allowed to cool slowly.

Conductivity (interior Heat Recorded)—*Results.*—The gravel specimens were the only ones that attained an interior temperature equal to the furnace temperature.

It is surprising to note that cinder came next to gravel in the amount of interior heat recorded, for cinder concrete is supposed to be an effective fire resistant.

Prof. Woolson says: "These tests prove that 2 or 3 in. of concrete properly mixed, tamped and set, will resist a fierce conflagration for hours without permitting a serious temperature rise upon the opposite side."

Effect of Heating on one Side only.—The strength of the 6 in. by 6 in. by 14 in. specimens heated on one face is in marked contrast to those heated on all sides, and shows that under that condition the concrete retains a large part of its strength even after five hours of exposure to a temperature of 1500° F.

Loss in Strength due to Excessive Heat.—The limestone lost about 50 per cent of its strength when heated two hours, and the trap about 55 per cent. The difference was probably accidental, for it is generally supposed that trap is superior to limestone in resisting heat. They both lost 68 per cent of their strength in five hours, when heated on all sides with no radiation permissible.

In a paper read by Mr. Frank B. Gilbreth before the American Society of Mechanical Engineers (1910), entitled "Fires and their Prevention" the author urged the use wherever possible of concrete and reinforced concrete in order to afford fire protection. The National Board of Fire Underwriters in Chicago have been carrying out some severe tests on various building materials, including reinforced concrete, and have subjected this material to a very high temperature for a period of two hours, after which water was applied. The tests were carried out by Mr. Richard L. Humphrey, who says:—

"The fact brought out most clearly by these tests is the low rate of heat transmission of Portland cement concretes and mortars. This is one of the desirable qualities in buildings intended for 'fireproofing purposes'.

One of the authors, in a paper read before the Concrete Institute,[1] described a series of tests carried out by him in order to ascertain the effect of excessive heat upon concrete and steel reinforcement.

A few extracts from this paper are here given:—

Effect of High Temperature upon Concrete.—"In order to obtain some reliable data regarding this important matter the author determined to carry out two series of tests, one dealing with concrete that had only had a short set, the other with concrete that had had at least two months' set. Twenty-four briquettes were made and placed in a boiler flue where the temperature averaged 850 to 900 degrees F. The briquettes were as follows:—

3 neat briquettes, placed in flue for 14 days.
3 ,, ,, ,, 28 ,,
3 sand and cement (3 to 1) briquettes in flue for 14 days.
3 ,, ,, ,, ,, 28 ,,
3-2¾ in. cubes, placed in flue for 14 days.
3-2¾ ,, ,, ,, 28 ,,
3-2¾ ,, (3 to 1) sand and cement, in flue for 14 days.
3-2¾ ,, ,, ,, ,, 28 ,,

At the same time that the above briquettes were placed in the boiler flue a similar number were placed in water. The experiments were carried out for the author by Messrs. G. & T. Earle, Ltd., cement manufacturers, Hull. The results were as follows:—

Tests of Cements Used in Experiments.—Cubes and briquettes in water for 7 and 28 days, after being in air for 14 days.

"[1] Reinforced Concrete Chimney Construction," paper by E. R. Matthews, Concrete Institute, January, 1910.

	7 days lb.	28 days lb.
Neat tensile strain, 1 in. section, moulds filled by thumb pressure	620	740
Neat tensile strain, 1 in. section, moulds filled by brass rammer	720	850
Sand tensile strain, 1 in. section, moulds filled by brass rammer	310	460
	tons.	tons.
Neat compression strain, $2\frac{3}{4}$ in. cube, moulds filled by brass rammer	26·5	33·6
Sand compression strain, $2\frac{3}{4}$ in. section, moulds filled by Klebb hammer	11·5	14·1

	Tons.	Average.
3 neat cubes in air, 14 days; in flue, 14 days	48·2, 50·0, 50·0	49·4 tons, on $2\frac{3}{4}$ in. cube
3 neat cubes in air, 14 days; in flue, 28 days	49·6, 53·6, 54·7	52·6 ,, ,, ,, ,,
3 sand cubes in air, 14 days; in flue, 14 days	12·4, 14·0, 14·6	13·3 ,, ,, ,, ,,
3 sand cubes in air, 14 days; in flue, 28 days	13·6, 13·8, 14·7	14·0 ,, ,, ,, ,,
	lb.	
3 neat briquettes, in air 14 days; in flue, 14 days	880, 890, 900	890 lb. on 1 inch section
3 neat briquettes, in air 14 days; in flue, 28 days	955, 990, 1,080	1,008 ,, ,, ,, ,,
3 sand briquettes, in air 14 days; in flue, 14 days	110, 110, 120	113 ,, ,, ,, ,,
3 sand briquettes, in air 14 days; in flue, 28 days	85, 100, 130	105 ,, ,, ,, ,,
Temperature, 850° to 900° F.		

Fire-box Test:—The author then had a fire-box made of concrete composed of 3 parts of ironworks clinkers (screened and broken to pass through a 1 in. ring), 2 parts pit sand, and 1 part cement. After a set of thirty days this fire-box was used, the result being that three or four very fine cracks appeared, which were undoubtedly caused by the sudden extraction of the moisture in the concrete. Apart from these cracks, the concrete remained in good condition for two months, in spite of the fire-box being used daily. The face of the concrete which came in actual contact with the fire daily was just as good at the expiration of two months as on the day the fire-box was first used. The cement used in this and the block tests described later was supplied by Messrs. Robson & Sons, of Hull, particulars of which are as follows :—

	April 15.	April 23.	April 29.
Neat cement at 7 days . .	550 lb.	647 lb.	631 lb.
,, ,, ,, 28 ,, . .	752 ,,	821 ,,	805 ,,
3 sand ; 1 cement at 7 days .	191 ,,	207 ,,	215 ,,
3 ,, 1 ,, ,, 28 ,, .	269 ,,	286 ,,	250 ,,
Le Châtelier test, 2·5 mm. .		0·75 mm.	3·0 mm.
Residue on 76 × 76 sieve .	0·6 per cent	0·5 per cent	0·4 per cent
,, ,, 180 × 180 sieve .	17·0 ,, ,,	16·0 ,, ,,	15·0 ,, ,,

ANALYSIS OF CEMENT.

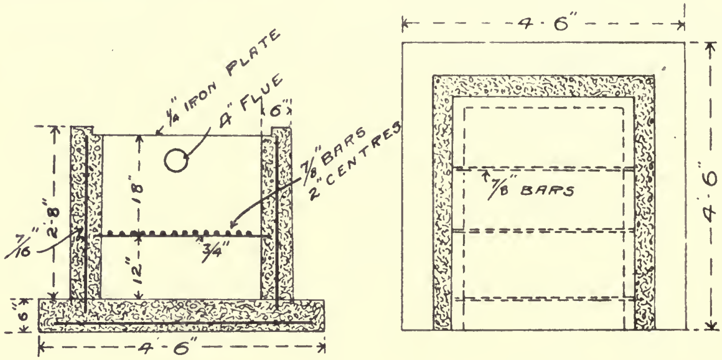
Loss on ignition	1·36 per cent.
Insoluble residue	0·52 ,, ,,
Silica	21·17 ,, ,,
Alumina	7·68 ,, ,,
Oxide of iron	3·64 ,, ,,
Lime	62·92 ,, ,,
Magnesia	1·21 ,, ,,
Sulphuric anhydride . . .	1·18 ,, ,,
Alkalies	0·32 ,, ,,

The quantity of water used in the mixing of the concrete was 10 per cent. The fire-box, was made on 4 June, and left in the open until it was used on 3 July.

Its construction is shown in Fig. 112, and it may be briefly described as follows :—

It consists of reinforced concrete bottom and three sides, with $\frac{1}{4}$-in. iron plate top supported by three $\frac{7}{8}$ in. diameter round bars. The slab forming the bottom of the fire-box is 4 ft. 6 in. sq., 6 in. thick, and reinforced on the underside by $\frac{7}{16}$

in. diameter round bars and clips; these have a covering of concrete on the underside of $1\frac{1}{2}$ in. Upon this slab the three sides of the box are erected; these are each 6 in. thick and 2 ft. 8 in. in height, and are reinforced in the centre with $\frac{7}{16}$ in. diameter bars. At the back of the box is a 4 in. flue. The



Section. Plan.

Fig. 112.

fire is supported by $\frac{7}{8}$ in. diameter loose bars, which have spaces of 2 in. between them, and which rest on three $\frac{3}{4}$ in. diameter round bars. It should be observed that this test is a most severe one, as the actual fire came daily in direct contact with the concrete.

Block Tests.—The author next determined to test concrete that had had two months' set and upwards, and for this purpose he had three blocks of concrete made, each being reinforced; the sizes and aggregates of the blocks were as follows:—

Sizes of blocks.—

12 in. × 12 in. × 4 in. thick,
12 ,, × 12 ,, × 4 ,, ,,
12 ,, × 12 ,, × 6 ,, ,,

Aggregates.—The first two blocks were composed of 3 parts of ironworks clinkers (screened and broken to pass though a 1 in. ring), 2 parts of pit sand, and 1 part Portland cement. The 6 in. block was composed of 3 parts of broken stone (broken to pass through a 1 in. ring), 2 parts of pit sand, and 1 part Portland cement.

Reinforcement.—The blocks were reinforced with $\frac{7}{16}$ in. bars and clips, and each block was an exact reproduction of a portion of a floor panel.

Ten per cent of water was used in mixing the concrete; the blocks were kept in air for two months and one day, and were then placed in boiler flues on 31 July, 1909, the temperature (average) being about 1250° F. during the whole of each day except Sundays, and falling at night and on Sundays to 260° F. at lowest. They were kept in these temperatures for twenty-one days, and when taken out of the flues and carefully examined the blocks were found to be quite unaffected by this great heat, except that one of the blocks, which was 4 in. thick, showed a slight hair crack on the face. No sign whatever of damage or cracking occurred on the other blocks, and they were in as good condition as when placed in the flues. The reinforcement in the blocks was also quite unaffected. In addition to these blocks, the author had other pieces of concrete—5 to 1, and of the same composition, but which in one case had had an eighteen months' set and in the other a six months' set—placed at the back of the fire-box before referred to, and resting on the fire-grate just below the flue-opening. These blocks came daily into contact with the actual fire—in fact, they were in the fire—and on being taken out at the expiration of fourteen days the condition of the concrete was not satisfactory; the blocks were not reinforced. The concrete fell to pieces on handling, and it is scarcely to be expected that it would do otherwise under such a terrific test.

The flue tests are, in the author's opinion, more satisfactory and reliable, as the actual flames did not reach the concrete. These block and fire-box tests were carried out for the author by Mr. A. Mitchell, one of the partners in the firm known as The Chain Concrete Syndicate, of Leeds. They were carried out at the works of this firm at Pudsey.

Further Tests and Examinations.—The author then determined to obtain some information regarding the condition of the concrete and reinforcement in an existing reinforced-concrete chimney, and he is greatly indebted to Mr. H. K. G. Bamber, F.I.C., of the Associated Portland Cement Manufacturers, who kindly supplied him with some most useful information on this subject.

Mr. Bamber informed the author that in order to ascertain if concrete deteriorated through the effects of heat, he inspected on several occasions the face of the concrete shell inside the reinforced-concrete chimney at the works of his firm at Northfleet, and he failed to observe any signs of deterioration. Not contenting himself with this, he subjected the ash in this chimney on various dates, to an analysis, and compared this with an analysis of the ash taken from the base of a brick Custodis shaft, also at his firm's works at Northfleet. The comparison is most interesting; it will be noticed that there is very little difference in the percentage of insoluble residue present, which residue would, of course, include any portions of sand that might, from deterioration, be coming away from the inside of the inner shell of the concrete chimney. The result of Mr. Bamber's analyses is set forth in the table on page 101.

Bevan's Works Chimneys, samples of flue dust taken from the Custodis (brick) and Weber (reinforced concrete) shafts:—

These residues are also periodically examined microscopically, and no perceptible difference has as yet been found.

Again, Messrs. G. & T. Earle, Ltd., inform the author that in order to ascertain the effect of heat upon concrete they placed a block of concrete (which had had several months' set) in one of their boiler flues for some months, the temperature in the flue being about 700° F. The block came out quite unaffected.

Effect of High Temperatures upon Steel Reinforcement.—The reinforcement in the blocks which the author has previously referred to was, as already stated, none the worse for being placed in a temperature of 1250° F. for twenty-one days; it was in precisely the same condition after the test as before.

Then Mr. Bamber very kindly had a small hole cut for the author in the base of the inner and outer shells of the reinforced-concrete chimney at Northfleet (see Figs. 113 and 114) for the purpose of exposing a portion of the reinforcement. These were made just over the flue opening where the heat would be most likely to affect the concrete. The result was most satisfactory, the steel being in as good condition as when first inserted in the concrete.

Date, 1908.	Weber Shaft.	Custodis Shaft.	Remarks.
October 17	13·60	—	
,, 19	14·10	—	
,, 20	14·60	17·90	
,, 21	16·30	—	
,, 22	16·70	—	
,, 23	17·06	17·20	
,, 24	13·68	—	
,, 26	15·90	—	Examined residue —both shafts exactly the same.
,, 27	16·40	—	
,, 28	16·10	18·62	
,, 29	16·84	—	
,, 30	16·90	18·44	
,, 31	17·70	—	
November 3	16·66	17·08	
,, 4	15·40	—	
,, 5	12·60	—	
,, 7	13·60	—	
,, 9	16·90	—	
,, 10	16·90	18·50	
,, 11	13·80	—	
,, 12	14·80	—	
,, 13	16·80	13·80	
,, 16	15·30	—	
,, 17	14·80	12·30	
,, 18	12·46	—	
,, 19	12·26	—	
,, 20	13·60	12·90	
,, 21	18·00	—	
,, 23	16·20	—	
,, 24	16·20	16·68	
,, 25	19·80	—	
,, 26	16·90	—	
,, 27	15·64	16·70	
,, 28	16·04	—	
,, 30	19·40	—	
December 1	19·66	17·14	Examined residue from both shafts—fine silica as from slurry.
,, 2	21 48	—	
,, 3	16·05	—	
,, 4	14·66	16·36	
,, 5	16 20	—	
,, 7	18 86	—	
,, 8	17·20	17·30	

Mr. Bamber describes his experiment thus: "I enclose you two photographs which will illustrate the experiments. The one with the square opening showing the hole at the back represents a photograph taken of the outside surface of the chimney, and the wall in which the smaller hole appears re-

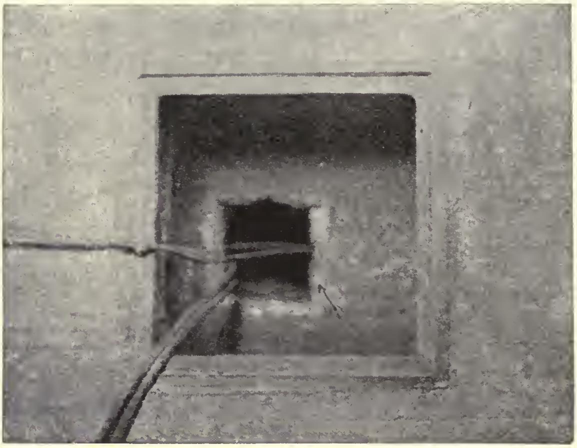


Fig. 113.

Fig. 114.

presents a $4\frac{1}{2}$ in. concrete lining inside the chimney, there being a 4 in. air space between the outer shell and this inner lining. A hole was cut through the inner lining for convenience during inspection, and in doing so we came across a portion of the steel reinforcement in this concrete. This has been in place for nearly two years, and has been subjected to a temperature inside the chimney of 500° to 600° F., and about 200° to 300° F. on the outside of the lining, i.e. in the air space. The concrete was carefully chipped away from the reinforcement, and upon careful examination, personally, I found that the concrete was adhering tightly to the reinforcement, which was in splendid condition. I marked the reinforcement lightly with the chisel, exposing the clean metal, and the white spots on the photograph are the reflection of light upon this metal. This second photograph was taken with a magnesium light close up to the inner opening. This seems to be an excellent example of the use of reinforced concrete under very trying conditions, and I would add that the internal examination of the chimney, which was done by means of a powerful searchlight thrown upon the interior surface from the bottom, showed this surface to be in as perfect condition as the outside."

Author's Conclusions.—The author's experiments led him to the following conclusions :—

1. That neat cement behaves better under great heat than out of it. The average compressive strength of $2\frac{3}{4}$ in. cubes placed in flue for twenty-eight days, after having remained in air for fourteen days, was 52·6 tons on the cube compared with 33·6 tons when placed in water for a similar period ; the tensile strength per square inch of the briquettes being 1008 lb. compared with 850 lb. in the water test.

2. That concrete (3 parts standard sand to 1 part Portland cement) if not well set behaves very badly under heat. The average tensile strength of briquettes 1 in. section (3 to 1) placed in flue for twenty-eight days, after being in air for fourteen days, was 105 lb. compared with 310 lb. at seven days, and 460 lb. at twenty-eight days in water.

3. That if the concrete has had at least a two months' set before heat is applied a temperature of 900° F. will not affect it in the least. This might be taken as the safe temperature in reinforced-concrete chimneys.

4. That the 3 to 1 specimens in these tests giving such poor results point to the necessity of having no voids whatever in any concrete work, it being well known that 3 to 1 briquettes

made of standard sand contain a considerable quantity of air spaces.

5. That only allowing the specimens fourteen days before subjecting them to the heat is far too short a period in actual work. The greatest advantage is obtained by letting the heat get at the concrete after the longest possible time has been given for the cement to set.

6. Concrete mixed with 10 per cent of water (which should be the maximum in work of this class) would contain only about 1 per cent of free water after a two-months' set. Heat should not be applied to concrete until the latter has had a two-months' set; a longer set, say three months, would be preferable.

The block tests carried out by the author were a most reliable guide as to the behaviour of concrete under conditions similar to that existing in a reinforced-concrete shaft. They were even subjected to a much more severe test than the inner or outer shell of a reinforced-concrete chimney would ever be subjected to, and in spite of that they came out unaffected. They were exposed to the heat on all sides, while the concrete in a chimney would only be attacked on one side. The temperature varied from 1250° F. during the day, to 260° F. during the night, and they were kept in these temperatures for twenty-one days.

If concrete will successfully withstand the severe test here indicated, it will much more easily withstand the conditions to which it would be subjected if forming the lining of a reinforced-concrete shaft.

2. Reinforced-concrete Tall Chimney Construction.—In the United States, over 1000 reinforced-concrete chimneys have been erected during the past eight years by one firm alone—the Weber Company of Chicago. In the British Isles not more than a dozen examples are to be found. The authors intend to give a brief description of two typical American chimneys, and two British. Before doing so, however they would enumerate the advantages of using this material in chimney construction, which are as follows:—

(*a*) Economy in cost (there being a saving of 25 to 30 per cent as compared with brick shafts).

(*b*) Saving of space.

(*c*) Less weight than brick shaft.

(*d*) Greater stability.

(*e*) Freedom from repairs.

(*f*) Rapidity of execution of work.

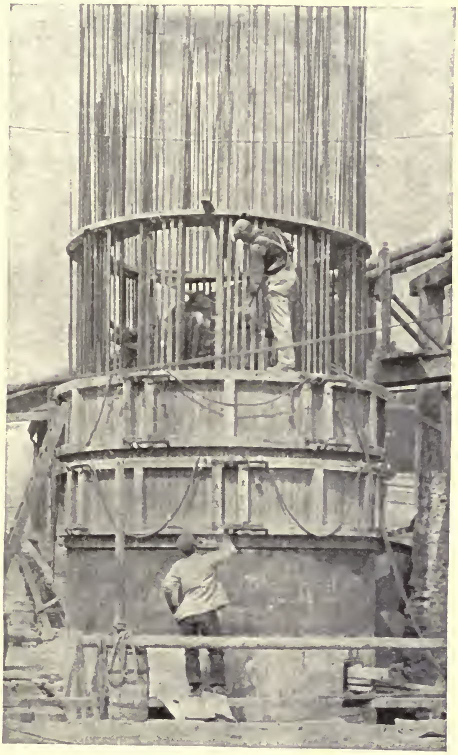


Fig. 115.

These advantages were set out in a paper read before the Concrete Institute in January 1910.[1]

Typical Examples :—The typical examples which the authors propose to refer to are :—

[1] "Reinforced-concrete Chimney Construction," by E. R. Matthews, Concrete Institute, January, 1910.

1. Chimney at Tacoma, Wash., U.S.A.
2. Chimney at Butte, Mont., U.S.A.

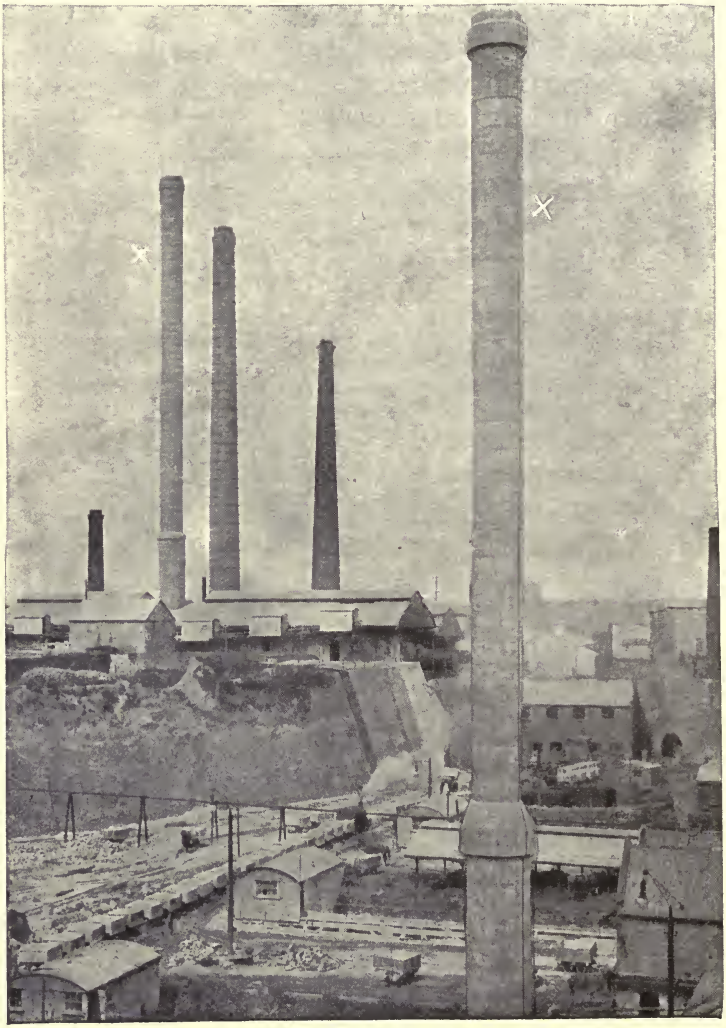


Fig. 116.

3. Chimney at Messrs. Lyle & Sons' Works, London, E.,
4. Large chimney at Northfleet, Kent.

The latter is illustrated in Figs. 115 and 116.

Illustrations are also given of a reinforced-concrete chimney at Indianapolis, Ind., U.S.A., and of one at New Bedford, Mass., U.S.A., the former being 200 ft. high and 12 ft. internal diameter, the latter 200 ft. high by 8 ft. 6 in. in diameter.

3. Effects of Frost on Concrete.—Many experiments have been made in order to ascertain the extent to which excessive frost injures freshly mixed concrete, and with a view of discovering if concrete frozen and again thawed before setting, is permanently reduced in strength, and if so, to what extent.

With regard to the latter point, it may be stated that until recently it was generally be-

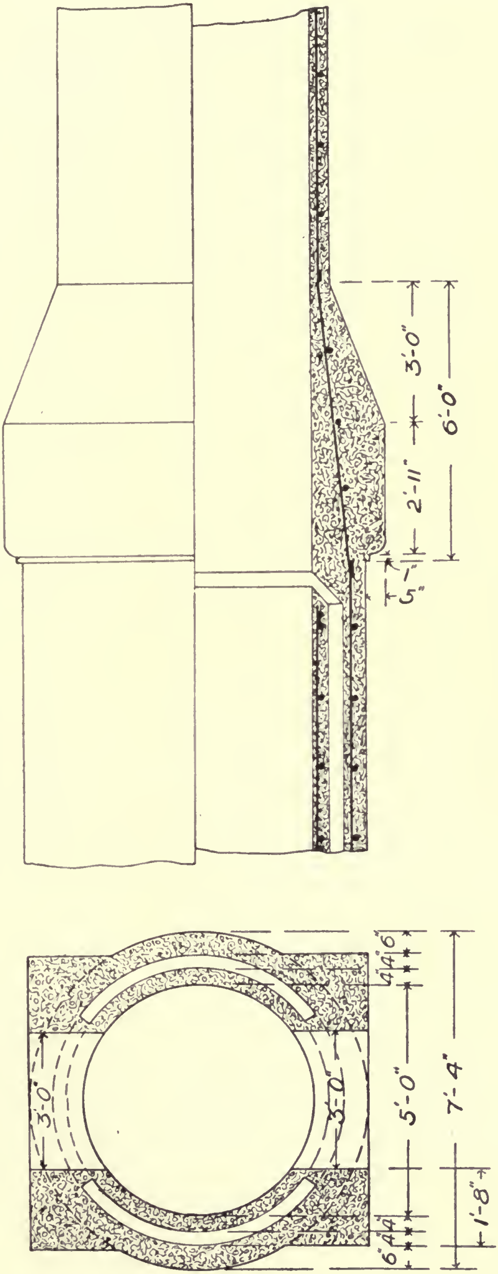


Fig. 117.

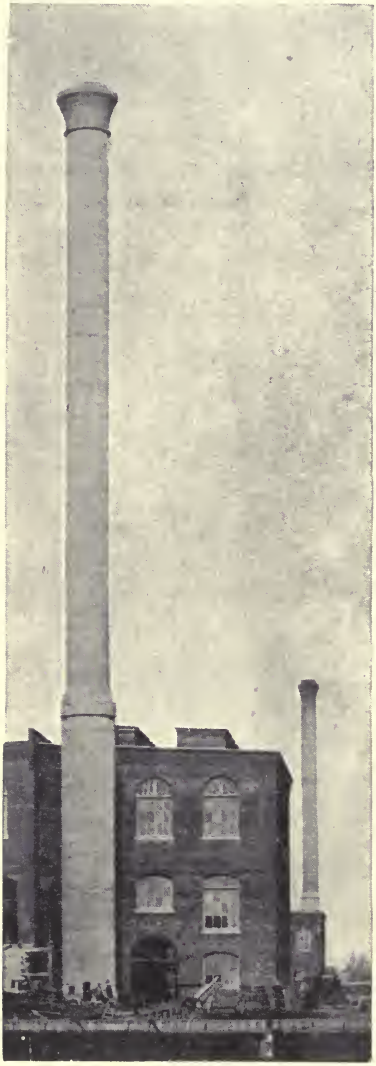


Fig. 118.

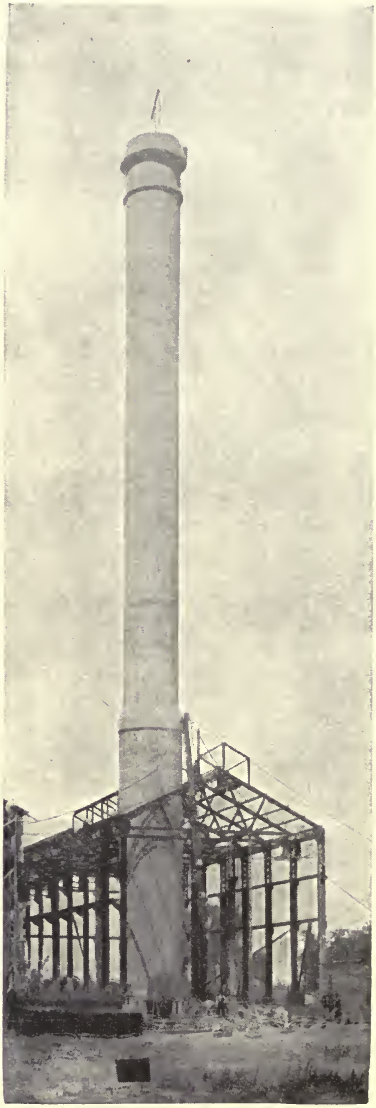


Fig. 119.

Particulars Relating to Four Typical Chimneys.

Location of Chimney.	Height in feet, above ground level.	Inside diameter in feet.	Thickness of Shell in Inches. Inner Shell.	Single Shell.	Outer Shell.	Reinforcement.	Remarks.
Tacoma, Wash., U.S.A.	300	18	5	7	9	Vertical bars and horizontal rings, the latter being 3 ft. apart in the outer shell, and single thickness of the chimney, and 1 ft. 8 in. apart in the inner shell.	Double shell extends about one-third of height of chimney. Foundations of chimney were 39 ft. 6 in. square. Shaft erected in 1905.
Butte, Mont., U.S.A.	350	18	5	7	9	Vertical bars $1\frac{1}{4}$ in. × $1\frac{1}{4}$ in. × $\frac{3}{16}$ in. T's, and horizontal rings 1 in. × $\frac{1}{8}$ in. T's, spaced at 18 in. centres.	This shaft has been erected at the Butte Reduction Works, Butte, Mont.
Works of Abraham Lyle and Sons, Ltd., London, E.	261	20	5	8	12	Vertical bars $1\frac{1}{4}$ in. × $1\frac{1}{4}$ in. × $\frac{3}{16}$ in. T's and horizontal rings of $\frac{5}{8}$ in. steel spaced at 18 in. centres.	The aggregate used was clean-washed Thames sand $3\frac{1}{2}$ to 1. No stone whatever.
"Bevan's Works," Northfleet, Kent.	247	8 ft. 6 in	4	9	12	ditto.	At Works of the Associated Portland Cement Manufacturers, Ltd.

lieved that freshly-mixed concrete which had frozen and again thawed before setting was rendered useless, but the experiments carried out by Professor H. Burchartz at the Royal Testing Station of Gross-Lichterfelde, Berlin, in 1910, for the purpose of testing this point, seem to refute this idea. These tests show that if concrete is allowed to warm up again to about its original temperature before setting, it is little affected by the frost. If the temperature when laying the concrete is low, the setting will, of course, be considerably retarded. If the freezing continues for several days, the hardening of the concrete is greatly delayed. Prof. Burchartz found that the twenty-eight days' strength of 1 to 3 briquettes was only 40 per cent of its normal value after three days' freezing (followed by thawing) if mixed comparatively dry, and 62 per cent if mixed wet, seeming to indicate that a "wet mixture" is not so much affected by frost as a "dry mixture".

The following particulars regarding a series of experiments carried out for the purpose of ascertaining the action of frost on cement and cement mortar are taken from a paper read before the American Society of Civil Engineers in 1909.[1] The conclusions arrived at by the authors of this paper should be noted.

This paper describes in detail a series of experiments, extending over the past two years, made by the writers, in order to ascertain:—

The effects of frost, and alternate frost and thaw, on the tensile strength of cement and cement mortar when mixed with

(*a*) fresh water, cold or warm,

(*b*) sea-water.

The temperature below which it is detrimental to mix Portland cement concrete.

The Hull Cold Storage Company of Hull, England, kindly allowed the writers to use its refrigerating rooms in order to obtain the degrees of frost necessary for these experiments. The cement used was that manufactured by Robson's Cement Company, of Hull, and all the experiments were carried out at the laboratories of that firm in Hull, these being kindly placed at the disposal of the writers.

Cement used in Experiments.—The particulars of the cement used are as follows: Made on 28 January, 1907. Residues on a 5776 sieve (that is, having 76 meshes per linear inch) = 0·5 per cent; residues on a 10,000 sieve (having 100 meshes per linear inch) = 2·0 per cent; residues on a 32,400 sieve (having 180 meshes per linear inch) = 11·5 per cent (showing that the cement was ground extremely fine). Specific gravity = 3·112; flour

[1] "The Action of Frost on Cement and Cement Mortar," by E. R. Matthews and J. Watson, American Soc. C.E., March, 1909.

= 54·5 per cent; Le Châtelier tests, expansion = 2·7 mm.; Faija bath test, cement pat hard and sound; time of set of neat cement with 25 per cent water: initial set = thirty-five minutes; permanent set = six hours, in a room kept at a temperature of 60° F.

Tensile Strains.—*Neat Cement.*—

7 days	= 685 lb. per sq. in.	(British Standard test 400 lb. per sq. in.)
14 ,,	= 787 ,, ,,	
28 ,,	= 875 ,, ,,	(,, ,, ,, 500 ,, ,,)

It will be seen that these results are well above the British standard tests.

Tensile Strains.—*One Part Cement and Three Parts Sand.*—Mixed with three parts by measure of sand, and with no hammering of briquettes into moulds, its mean strength was :—

7 days	= 200 lb. per sq. in.	(British Standard test 120 lb. per sq. in.)
14 ,,	= 277 ,, ,,	
28 ,,	= 333 ,, ,,	(,, ,, ,, 225 ,, ,,)

Chemical Analysis.	Percentage.
Insoluble residue . .	0·82
Silica	20·43
Alumina	9·10
Oxide of iron . . .	1·95
Lime	62·65
Magnesia	1·25
Sulphuric anhydride . .	1·38
Loss on ignition . . .	1·66
Alkalies and loss . . .	0·76
	100·00

The sea water used in these experiments was taken from the North Sea; the fresh water was drawn from the Hull Corporation mains. To obtain the soft water, the temporary hardness of this water was removed, the permanent hardness being from 3° to 4°.

The experiments herein will be compared with the tests just given, which will be referred to as the "normal tests".

Effects of Frost.—The effects of frost, and alternate frost and thaw, on the tensile strength of cement and cement mortar when mixed with (*a*) fresh water, cold or warm, (*b*) sea water; and the temperature below which it is detrimental to mix Portland cement concrete, were determined by the following experiments:—

Experiment A.—In this experiment the writers set out to discover the weakening effect, upon freshly mixed cement, of continuous light frost, temperature 29° F., and of heavy frost, 15° F. Nine briquettes were made with neat cement, 20 per cent of water, in the laboratory, the temperature of the air being 60° F. These were taken from the moulds twenty-four hours after gauging and placed in cold stores, temperature 29° F., and were broken at seven and twenty-eight days respectively, the average tensile strength being:—

At 7 days = 610 lb. per sq. in.

At 28 ,, = 905 ,, ,,

These results, compared with the normal tests, were as follows:—

Tensile strength, in lb. per sq. in.

Days.	Normal tests 24 hours in air at 60° F., in water remainder of time.	In air at 60° F. for 24 hours. then 29° F., for remainder of time.
7	685	610
28	875	905

In the seven days' test it will be observed that there is a decrease of 10·9 per cent in tensile strength, and an increase of 3·4 per cent in the twenty-eight days' test.

Experiment B.—Nine briquettes, made in the same manner as in Experiment A were placed at 60° F. Three were taken out at the end of two days, and placed in cold storage for twenty-five days; three more, at seven days, for twenty-one days, and the other three, at fourteen days, for fourteen days. All were broken at twenty-eight days, the result being as follows:—

Tensile strength, in lb. per sq. in.

Days.	Normal test in air, 60° F. for 24 hours, then in water.	In water at 60° F. for 2 days, then in air at 29° F.	In water at 60° F. for 7 days then in air at 29° F.	In water at 60° F. for 14 days, then in air at 29° F.
28	875	912	977	942

Experiment C.—Nine briquettes, made as before, were allowed to harden in air for seven and twenty-eight days at 60° F., the result being :—

Average tensile strength at 7 days = 443 lb. per sq. in.
,, ,, ,, 14 ,, = 525 ,, ,,
,, ,, ,, 28 ,, = 775 ,, ,,

Days.	7, 14, and 28 days in air at 60° F.	24 hours in air at 60° F., then in water.
7	433	685
14	525	787
28	775	875

Experiment C_1.—Sand and Cement Test (3 *to* 1). *Same as experiments A, B, & C.*

Days.	Normal test.	A.	B.	C.
7	200	163	—	220
14	277	270	—	250
28	353	342	B 2 317 B 7 305 B14 243	322

Experiment D.—Effect of Alternate Frost and Thaw.—The briquettes were allowed to remain for twenty-four hours under damp flannel, then in water for three days (60° F.) then in water in the cold stores (temperature varying from 29° to 60° F.). The briquettes were changed every three days.

Days.	Neat.	3 to 1.	Normal.	
			Neat.	3 to 1.
14	787	252	787	277
28	813	323	875	353

Experiment E.—This test was the same as A or D, but the briquettes were gauged with warm water; temperature 100° F.

Days.	Neat.	3 to 1.	A Test.	
			Neat.	3 to 1.
7	352	133	610	163
14	705	205	—	270
28	728	230	905	342

Experiment F (salt water immersion).—Nine briquettes were mixed with fresh water, and after twenty-four hours, were immersed in sea water, and broken at seven, fourteen, and twenty-eight days.

Days.	Neat. 20 per cent water.	3 to 1. 10 per cent water.	Normal.	
			Neat.	3 to 1.
7	770	242	685	200
14	742	278	787	277
28	812	360	875	353

Experiment H.—Nine briquettes were mixed with sea water, and after twenty-four hours under damp flannel, were immersed in fresh water for the remainder of the time.

Days.	Neat.	3 to 1.	Normal.	
			Neat.	3 to 1.
7	628	150	685	200
14	733	255	787	277
28	713	297	875	353

Experiment K.—Same test as A, but the briquettes were kept in a temperature of 15° F., in cold storage

K., Heavy Frost.			A., Light Frost.		Normal.	
Days.	Neat.	3 to 1.	Neat.	3 to 1.	Neat.	3 to 1.
7	405	57	610	163	685	200
28	595	145	905	342	875	353

The briquettes were taken from the cold stores to the laboratory, two miles away, and were broken in a temperature of 60° F., forty-five minutes after leaving the cold stores.

Experiment L.—The briquettes, twenty-four hours after gauging, were put in water at 60° F., for six days, then placed in the cold stores at a temperature of 15° F., for the remainder of the time.

Days.	Neat.	3 to 1.	Briquettes placed in air at 60° F. for a day or two after having been in cold stores at 15° F. for 28 days.		A.	
			Neat.	3 to 1.	Neat.	3 to 1.
28	700	217	875	315	905	342

Experiment M.—The briquettes were put directly into the cold stores at 29° F., for seven days.

Days.	Neat.	Normal-Neat.
7	480	685
28	595	875

Experiment N.—The briquettes were made with neat cement, and placed, some in air at a temperature of 60° F., and others in air at a temperature of 29·3° F. (2·7 degrees of frost). In fifteen minutes those in air at 60° F. were still soft, while those subjected to frost had just frozen hard at the expiration of that time. Briquettes mixed with sand and cement (3 to 1) were subjected to a similar test. At the expiration of fifteen minutes those in a temperature of 60° F. were still soft, while those in a temperature of 29·3° F. had just frozen hard; and at a temperature of 27° F. (5° below freezing-point, F.) were frozen very hard indeed at the expiration of that time.

Conclusions from the Foregoing Experiments.—These investigations have led the writers to the following conclusions:—

1. That light frost occurring twenty-four hours after the cement has been gauged, as indicated in Experiment A (3 degrees of frost, or thereabouts), is detrimental to freshly mixed Portland cement but only for a short time, and that at the end of twenty-eight days it has quite regained its normal strength. If the frost occurs immediately after the cement has been gauged, the effect is more detrimental, and would appear to be permanent (see Experiment M). A minimum quantity of water should be added in frosty weather.

2. That heavy frost (17 degrees of frost, or thereabouts), has a most injurious effect (permanent) upon freshly mixed cement (neat), and cement mortar, as shown in Experiment K.

3. That a light frost (3 degrees of frost, or thereabouts), as indicated in Experiment A, does not affect cement or cement mortar if it has attained two days' set previous to the occurrence of the frost (Experiments B, C, and D).

4. That the detrimental effect of light frost upon cement mortar (3 to 1) occurs more immediately than upon neat cement, but that cement mortar recovers from the ill effects of frost more rapidly than neat cement. At the end of fourteen days it has quite recovered (Experiment C).

5. That the mixing of cement or cement mortar with warm water (temperature, say, 100° F.), which is sometimes done in frosty weather, and has been recommended by some engineers,[1] has a permanently injurious effect upon cement and cement mortar. This will be seen by reference to Experiment E.

6. Experiment L shows that 17 degrees of frost for twenty-eight days does not kill the process of hardening in the briquette, but only delays it.

7. It would appear from Experiment N that it is detrimental to concrete to mix it when the temperature is below 29·3° F. (2·7 degrees of frost), that being the freezing-point of cement and concrete.

[1] "Minutes of Proceedings, Inst. C.E.," London, Vol. CXXXIV, p. 384.

CHAPTER VII.

RECIPROCATION, DUALITY, ABRIDGED NOTATION, AND PROJECTION.

Reservoir and Tank Construction.—In no class of municipal engineering work has reinforced concrete been so satisfactorily used as in the construction of reservoirs, tanks, and culverts, for which purpose it is admirably suited. It is (*a*) considerably cheaper than plain concrete, quite 15 per cent cheaper; (*b*) it can be readily constructed of any shape; (*c*) it has no joints; (*d*) it requires no repairs; (*e*) works can be rapidly executed in this material; (*f*) increases in strength with age; (*g*) possesses great hygienic value; (*h*) excellent material for reservoir bottoms in bad ground.

Before describing a few typical examples of reinforced concrete reservoirs, the authors would state in what respects American engineers differ in their methods of reservoir construction from English engineers.

1. In the first place, American reservoirs built of this material are usually circular in plan, in this country they are rectangular; frequently the length is about double the width.

2. It is becoming the growing custom in the United States in covered reservoirs, to make the concrete or reinforced concrete piers circular in plan. It being contended that they :—[1]

(*a*) Occupy less space.

(*b*) Use less material.

(*c*) Cost no more; alternative prices having been invited in some cases with this result. It is the custom in England to make the columns square in plan, and they are sometimes splayed out in the bottom length.

3. As already intimated in Chapter I a "wet mixture" of

[1] "Reinforced Concrete Reservoirs," by E. R. Matthews. "Concrete and Constructional Engineering," July, 1909.

concrete is used in reservoir and tank construction in America, a plastic mixture being used in this country.

4. The proportions of the ingredients differ considerably; our American friends using a 1 : 3 : 7 or 8 concrete, while we should not think of using a weaker mixture for the reservoir floor or bottom and walls than 1 : 2 : 4, and a roof covering of 1 : 3 : 6.

5. Groined arches are usually constructed over service reservoirs in America. These are somewhat expensive on account of the shape of the forms required, and do not possess any special advantage.

In this country a flat covering is adopted; usually 4 to 6 in. in thickness, and supported by reinforced beams.

The material is sometimes used throughout the entire structure, and at other times in the construction of the roof only. For example, the roof of the Louisville reservoir is of reinforced concrete; this is one of the finest modern American reservoirs, and has a capacity of 25,000,000 gallons. The cost per square foot for covering this reservoir was 2s. 6d.[1]

The typical examples, however, which will now be given, in most cases represent reservoirs constructed throughout of reinforced concrete, and are British examples.

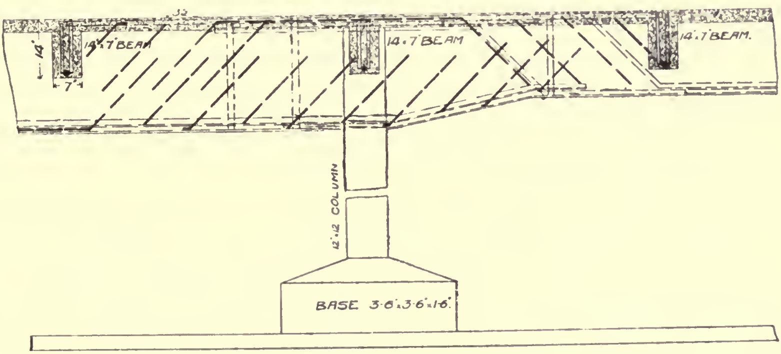


Fig. 120.

Dundee Reservoir.—Fig. 120 represents a reinforced concrete roof at the Stobsmuir Reservoir, Dundee. It well illustrates

[1] "The Use of Reinforced Concrete in Engineering Construction in America," by E. R. Matthews. "Jour. Royal Soc. of Arts," March, 1908.

the application of this material. The reservoir is 392 ft. in length by 145 ft. 8 in. (inside measurements) in width, and the flat roof is supported by reinforced concrete beams, and drains on to a channel. The work has been carried out by the Trussed Concrete Steel Co., Ltd., and designed on their Kahn system of reinforcement, which consists of cut bars arranged as shown in Fig. 3, p. 11.

The cross-section of these bars is diamond-shaped, with two horizontal flanges or wings, projecting at diametrically opposite corners.

Columns.—These are of reinforced concrete, the reinforcement consisting of four $\frac{1}{2}$ in. square rods, and $\frac{3}{16}$ in. wire. The size of the piers is 12 in. by 12 in. and the bases are 3 ft. 6 in. by 3 ft. 6 in. by 1 ft. 6 in. with splayed tops.

Beams.—The main beams are 20 in. by 8 in. and 27 in. by 10 in. in size; the former being reinforced by 1 in. diameter rods, 1 in. by 3 in. Kahn bars (inverted), and $1\frac{1}{2}$ in. by $\frac{1}{4}$ in. stirrups; the latter by $1\frac{1}{4}$ in. diameter rods, $1\frac{1}{4}$ in. by $3\frac{3}{4}$ in. Kahn bars, and stirrups.

The secondary beams are 14 in. by 7 in. reinforced by 1 in. by 3 in. Kahn bars and stirrups.

Roof Slab.—This is only $3\frac{1}{2}$ in. in thickness, and is reinforced with No. 8 expanded metal, and $1\frac{1}{4}$ in. by $3\frac{3}{4}$ in. and 1 in. by 3 in. inverted Kahn bars.

Roof Finish.—The surface of the roof is asphalted.

This work has been designed for, and carried out under the superintendence of, the waterworks engineer, Mr. Geo. Baxter, and the aggregate used consisted of 27 cu. ft. broken stone, to $13\frac{1}{2}$ cu. ft. sand, to 7 cu. ft. cement.

Water Softening Tank at Manchester.—(See Fig. 121). This tank is now being constructed at the Openshaw Works of Messrs. Armstrong, Whitworth and Co., Ltd., by Messrs Wm. Moss and Sons, Ltd. Their system consists of a number of looped shear members attached to bars of light steel joist section (see diagram) Fig. 10.

It is claimed that such an arrangement has an advantage over the ordinary small bars which have to be individually surrounded in concrete, and therefore result in the centre of gravity

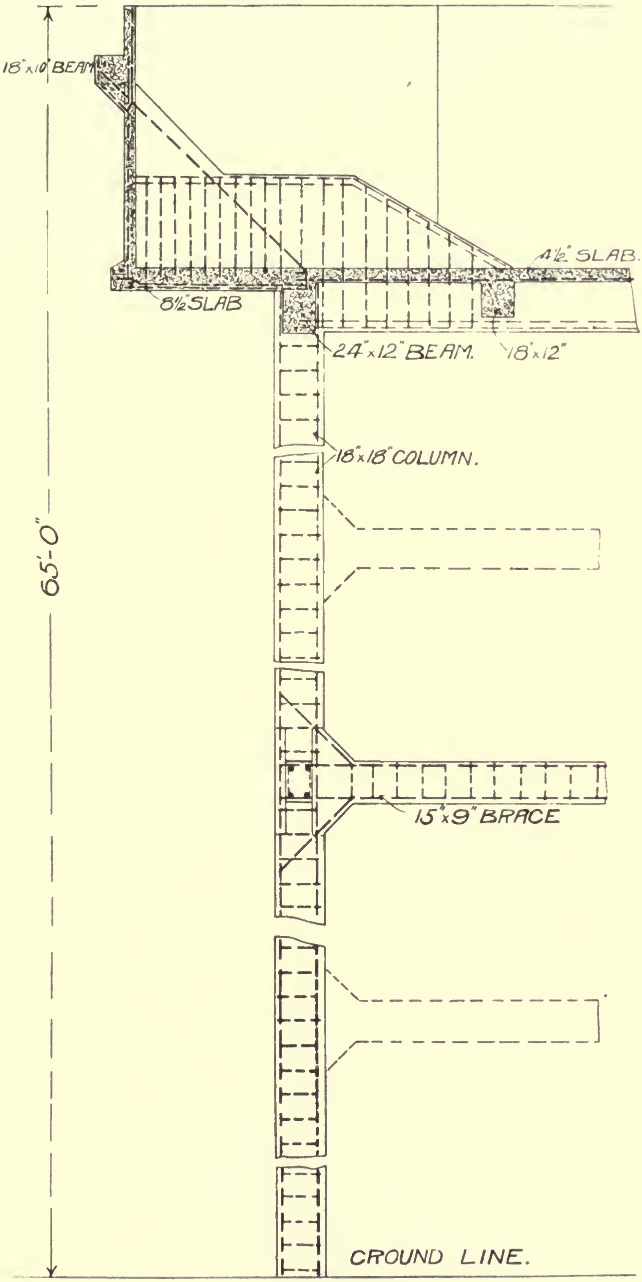

18"x10" BEAM
4½" SLAB.
8½" SLAB
24"x12" BEAM.
18"x12"
18"x18" COLUMN.
65'-0"
15"x9" BRACE
GROUND LINE.

FIG. 121.

of the total reinforcement being raised very considerably. The centre of gravity of the reinforcement in the Moss system is lower down than is the case where a large number of small units are employed.

With respect to the shear members, they can, of course, be of any length required. They are not simply threaded through holes in the reinforcing bars, but are wedged up so as to have a tight bearing on the web of the bars.

The special reason for this wedging up is that if the shear members were simply put through holes a certain amount of play would be bound to exist, and this would reduce the efficiency of the structure, as the slightest give in the shear member when under shear stresses would naturally mean a crack in the concrete beam.

The tank or water tower referred to has a capacity of 200,000 gallons.

This is an interesting piece of reinforced concrete work on account of the centre cylinder which reaches from the ground up to the top of the tank being under unusually high pressure. The top of the tank is 65 ft. above ground level. The walls of the tank are 4 in. in thickness, the floor slab being $4\frac{1}{2}$ in. in thickness, except a small area which is 8 in. in thickness. It is supported by 16 in. by 10 in., 18 in. by 12 in., and 24 in. by 12 in. reinforced beams. The tank is supported by 18 in. by 18 in. columns which are braced by 15 in. by 9 in. beams.

The centre cylinder has walls which diminish from 8 in. to 4 in. in thickness. The reinforcement of the braces consists of $\frac{3}{4}$ in. rods and $\frac{3}{16}$ in. wire ties, the latter having 9 in. pitch. The columns are each reinforced by four $\frac{3}{4}$ in. diameter rods, and $\frac{3}{16}$ in. wire ties. The walls of the tank are reinforced by $\frac{1}{2}$ in. diameter rods, and the floor by $\frac{1}{2}$ in. rods, $4\frac{1}{2}$ in. by $\frac{1}{2}$ in. flat bars, and 1 in. by $\frac{5}{32}$ in. flat stirrups.

Underground Tank at Dublin.—This tank (see Fig. 122) has recently been constructed at Parklands, Radeny, Co. Dublin. It is built on the Moss system of reinforcement. Its internal dimensions are width 22 ft. 9 in., length 50 ft. 5 in. The outer walls are 8 in. in thickness with splayed angles, and are reinforced with $\frac{3}{8}$ in. rods. The division wall is 9 in. in thickness, and is reinforced in a similar manner to the 8 in. wall. The floor is of plain concrete 6 in. in thickness. The roof is supported by reinforced concrete beams 12 in. by 6 in., which are

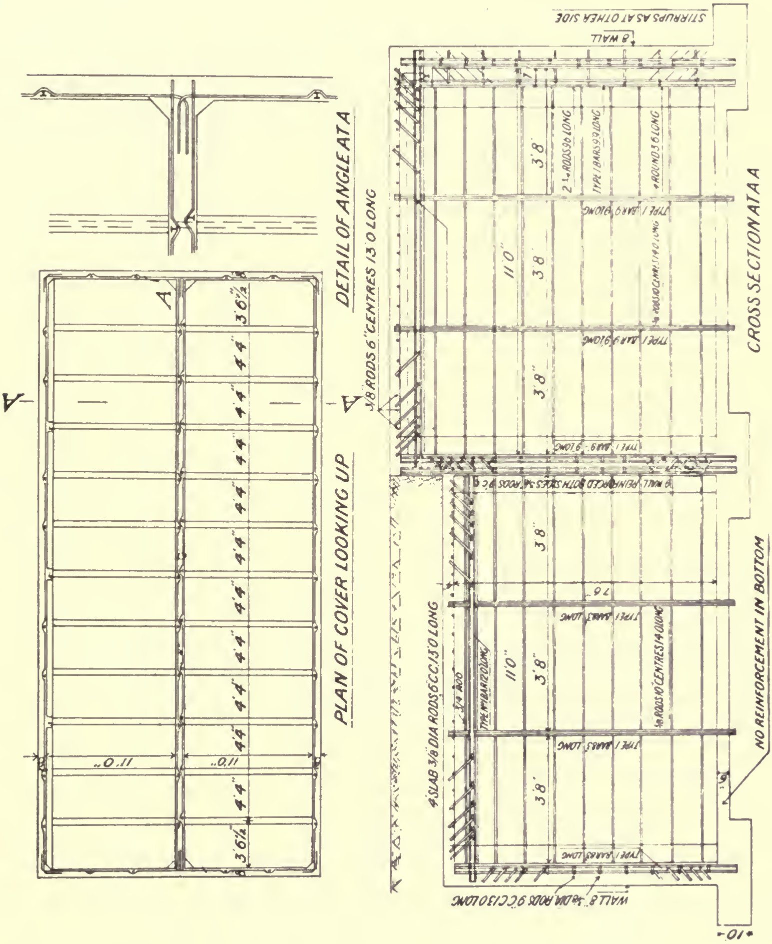


Fig. 122

reinforced by $\frac{3}{4}$ in. rods and stirrups. The slab is 4 in. in thickness, reinforced by $\frac{3}{8}$ in. rods spaced at 6 in. centres.

Liquor Tank.—Fig. 123 represents a reinforced concrete liquor tank, circular in plan, and supported by reinforced piers, octagonal in shape. The tank rests on reinforced concrete

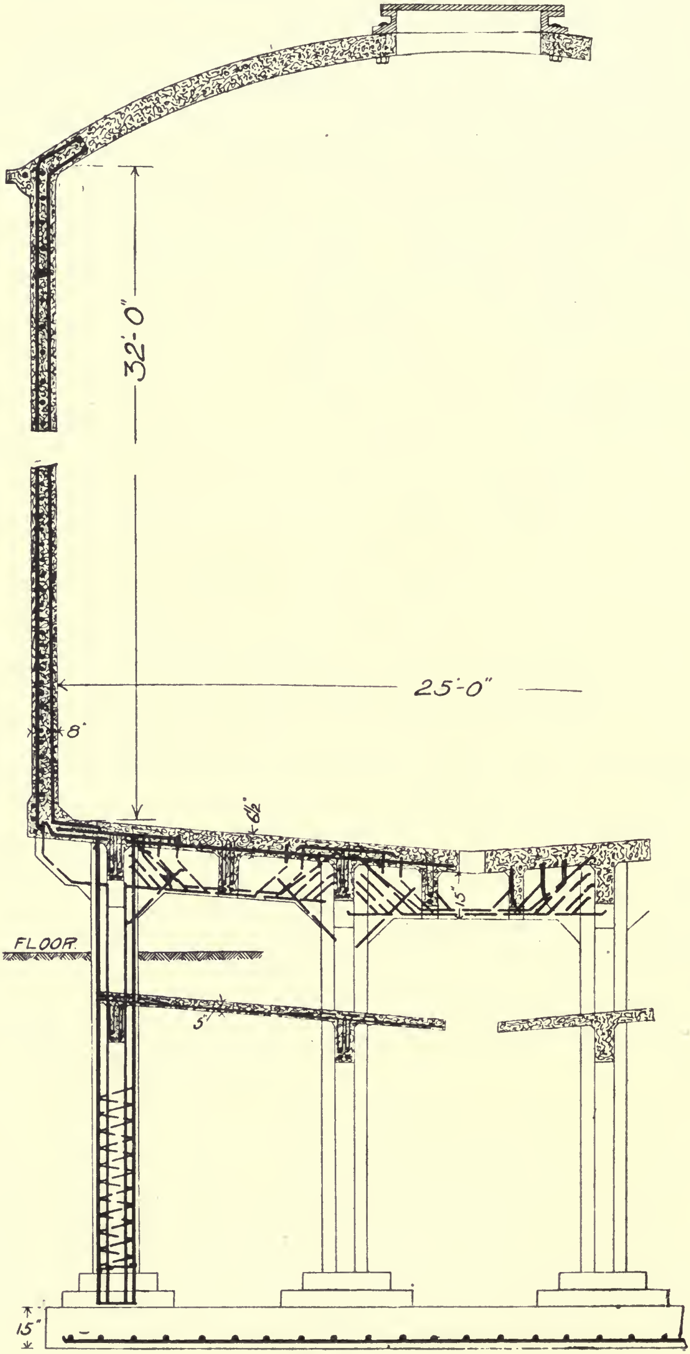


Fig. 123.

beams ; the floor is 6 in. in thickness, the walls 8 in., the domed roof 6 in. The dimensions are as follows: Internal diameter 25 ft., height 32 ft. This tank is intended to hold concentrated ammonia liquor.

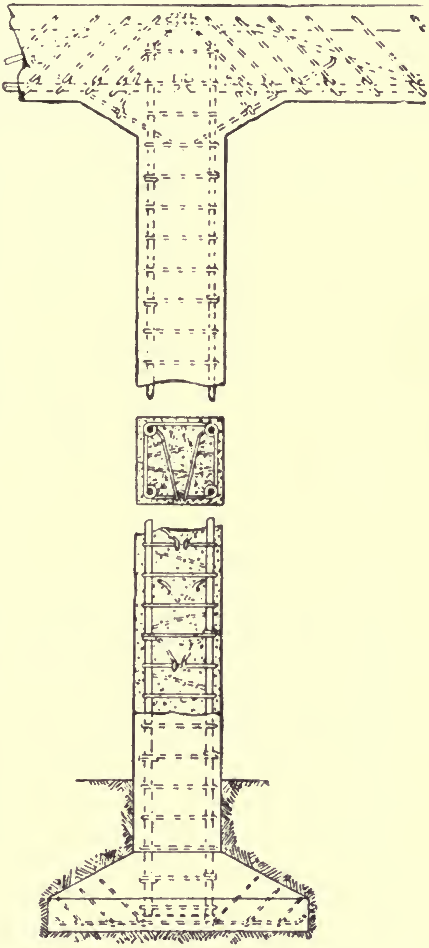


Fig. 124.

Investigations have been made with a view to discover the effects of hydrochloric acid and ammonia liquors on concrete; the effect was noted of boiling concrete blocks in acid solution until the projecting steel was corroded away. Where the concrete had been properly graded, mixed and consolidated, the penetration of the acid was quite insignificant.

Further experiments on the interior of tanks for concentrated hydrochloric acid, proved that a light coating of bitumastic solution penetrated the concrete, sufficient to form a skin impervious to this acid.

Ammonia liquor has a corrosive action on steel, and the original tanks which are now being substituted by reinforced concrete ones, were lined with blue brick, set in cement mortar, but in places the liquid penetrated through this, owing probably to defective joints, and seriously affected the steel.

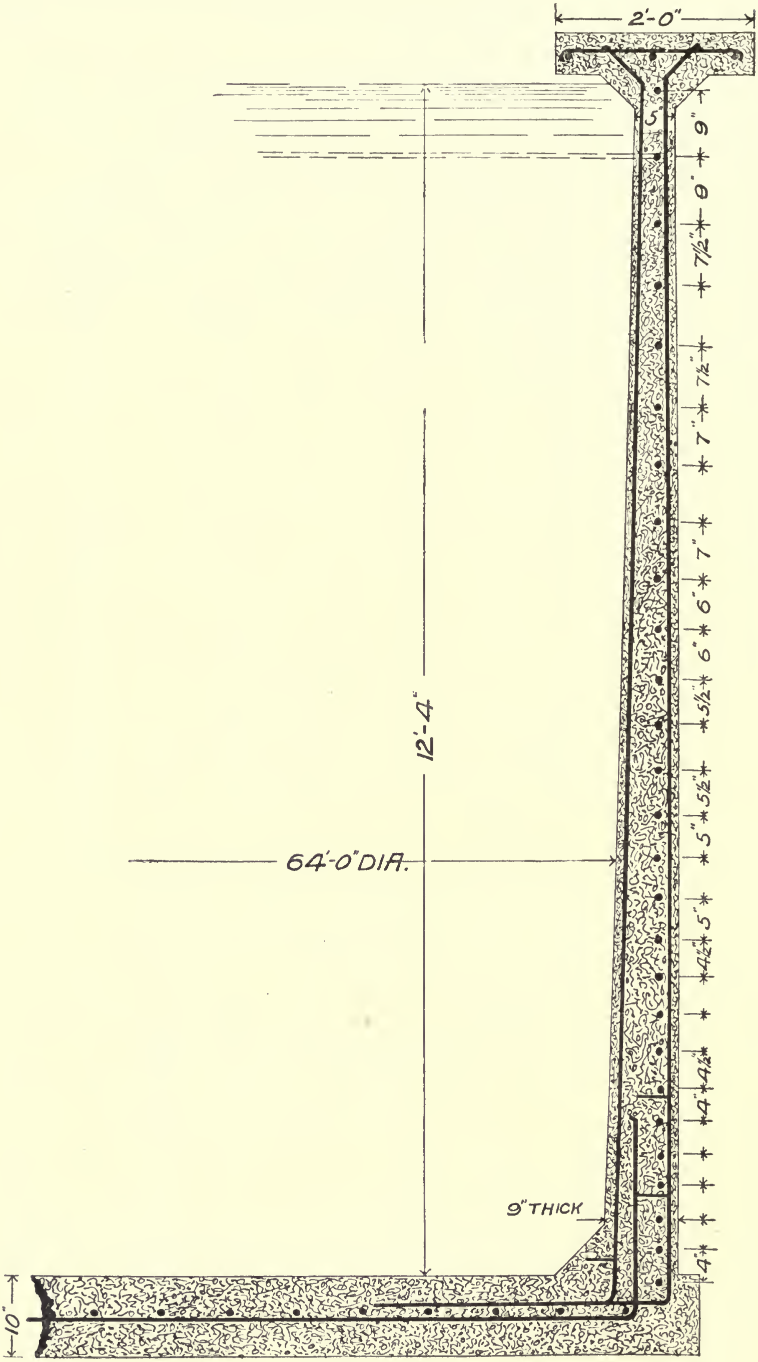


FIG. 125.

The experience of this firm warrants them in coating the interior of the reinforced concrete tank with silicate of sodium which combines with the lime in the concrete, and, properly applied, gives a skin like glass, hard and impervious.

The tank is being constructed for one of the largest chemical manufacturing firms in the United Kingdom.

The British Reinforced Concrete Engineering Co., Ltd., are carrying out this contract. Their system of reinforcement consists of trussed units for beams, floors, etc., made respectively with round and square bars, and patent stirrups which are usually fixed at an angle of 45 degrees (see diagram).

It is claimed that with this system a perfect mechanical bond

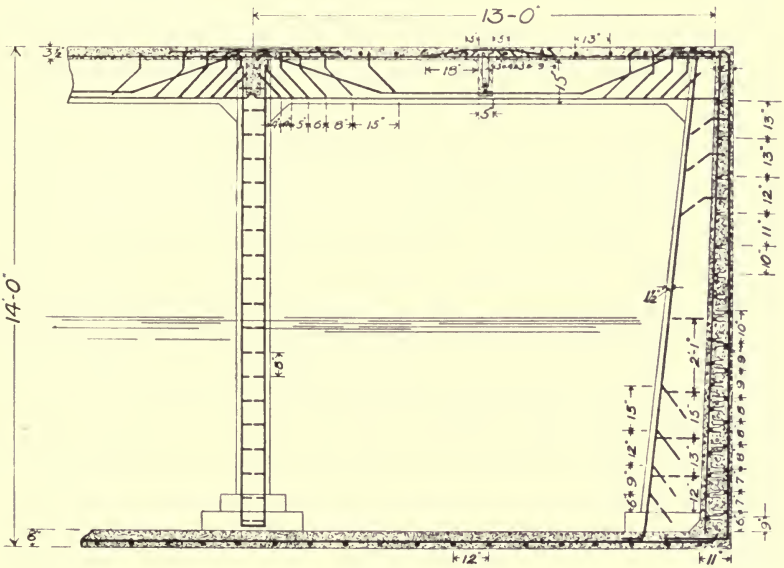


Fig. 126.

is assured, and that the stresses are distributed along the structure in such a manner that the steel and concrete act together in taking the load. The stirrups are made long enough to reach right up into the compressional area of the deepest beams.

Other reinforced concrete tanks constructed by this firm include the following four :—

Tank at Newton, near Wigan.—(Illustrated in Fig. 125.)

This is a circular tank of 64 ft. diameter, the maximum depth of water being 12 ft. 4 in. The walls diminish from 9 to 5 in.

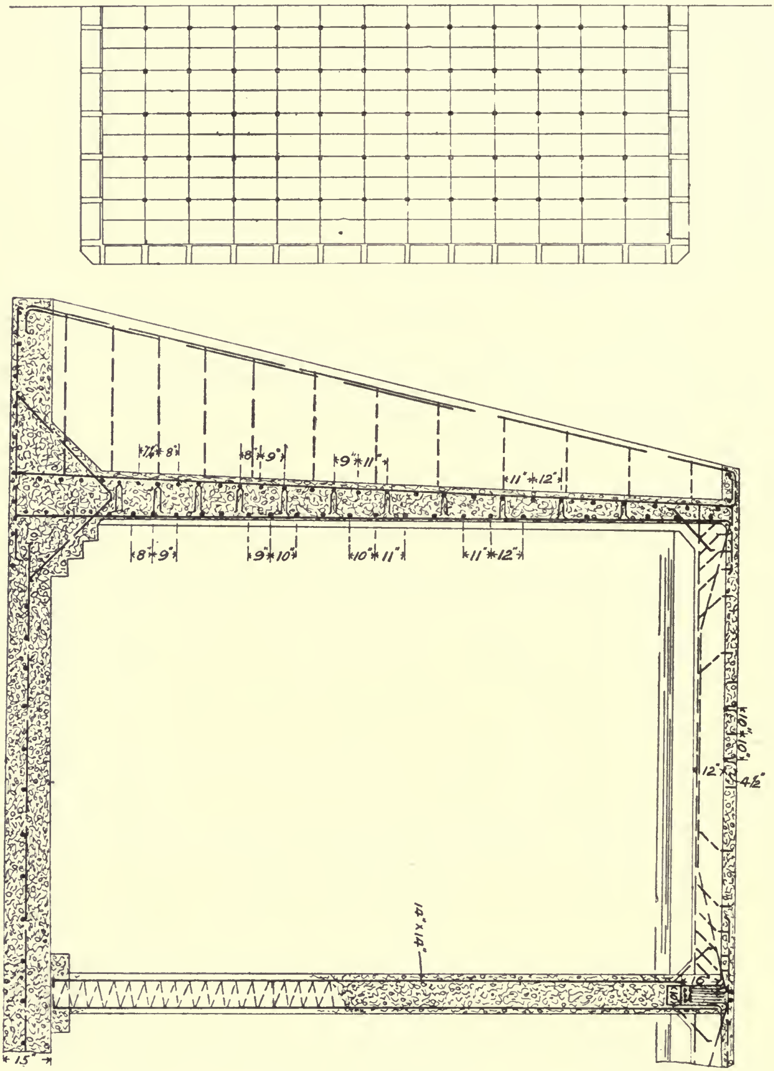


Fig. 127.

in thickness, and the floor is 10 in. thick. The reinforcement consists of $\frac{3}{8}$ in., $\frac{5}{16}$ in., $\frac{7}{8}$ in., $\frac{15}{16}$ in., and 1 in. diameter rods, and stirrups.

Covered Tank at Milnsbridge.—This is shown in Fig. 126. It is 14 ft. in depth, and of the shape illustrated. The floor is 6 in. in thickness, the roof slab $3\frac{1}{2}$ in. supported by beams 15 in. by 5 in. The walls diminish from 11 in. to 6 in. in thickness. The reinforcement consists of round bars, and stirrups.

Tank at Cheshire.—The construction of this is shown in Fig. 127. The reinforcement of each pier consists of four 1 in. round rods, and $\frac{5}{16}$ in. helicals. The piers are 14 in. by 14 in. The floor of the tank is 15 in. in thickness, and is reinforced by $\frac{3}{8}$ in. and 1 in. rods. The walls have buttresses, and the roof slab is supported by reinforced concrete beams.

Tank at Storage Warehouses, Trafford Park, Manchester.—Fig. 128 illustrates this. It is 16 ft. by 16 ft. and 5 ft. deep, and is supported by rolled steel joists. The walls vary in thickness from 5 to $3\frac{1}{2}$ in.; the floor is 5 in. thick, and the roof slab $3\frac{1}{2}$ in.

Covered Service Reservoir, Ipswich.—Fig. 129 represents the new service reservoir erected a short time ago for the Ipswich Corporation. It has been built to the designs of the water engineer—Mr. Hamlet Roberts, A.M.I.C.E.

The dimensions of the reservoir are as follows: length 300 ft., width 200 ft., depth 15 ft. It is constructed throughout of concrete, except the roof which is 6 in. in thickness and reinforced by expanded metal No. 8 gauge; the roof is supported on rolled steel joists resting on cast iron columns.

The concrete mixture was 6 to 1 (ballast and cement).

The reinforcement was supplied by the Expanded Metal Co., Ltd.

Circular Reservoir at Albion, Mich., U.S.A.[1]—This is represented in Fig. 130. The reservoir is constructed throughout of reinforced concrete. Its capacity is 100,000 gallons. It was built for the Gale Manufacturing Co., and is reinforced on the Kahn system.

Reservoir at Annapolis, Mo., U.S.A.[1]—The Naval Academy

[1] "Reinforced Concrete Reservoirs," by E. R. Matthews. "Concrete and Constructional Engineering," July, 1909, p. 163.

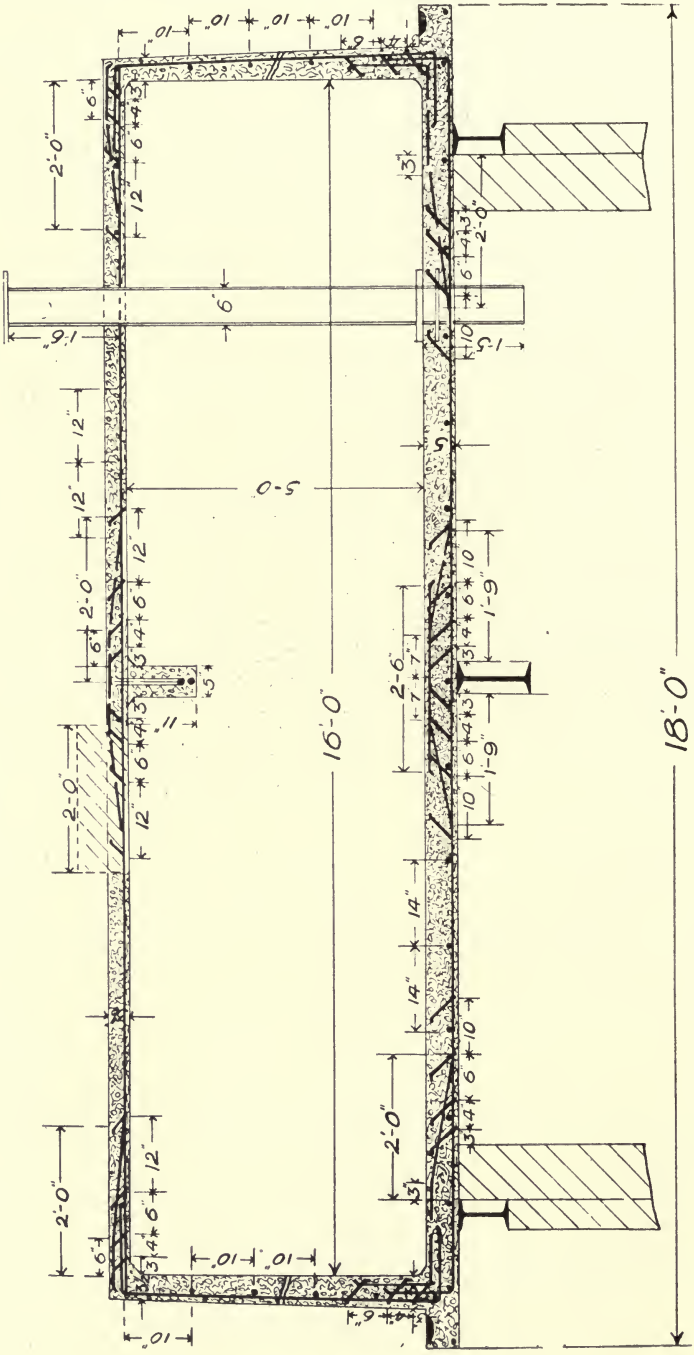


FIG. 128.

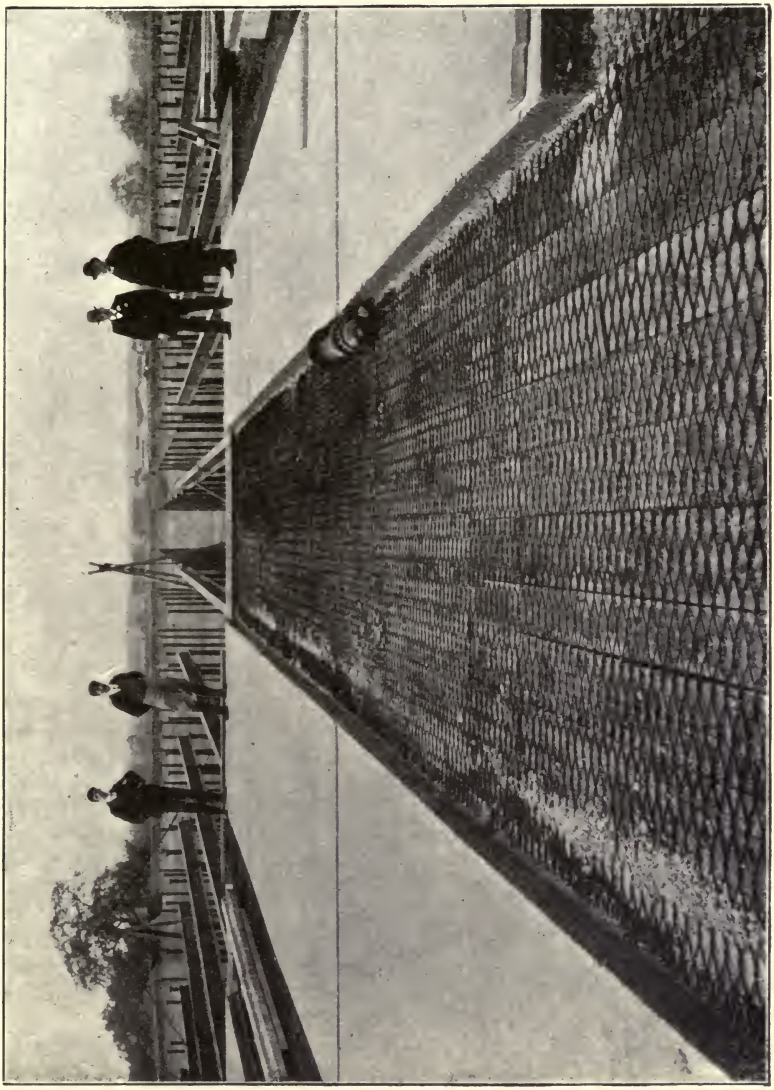


FIG. 129.

Reservoir at Annapolis is shown in Fig. 131. The reinforcement in this case is on the Kahn system.

Reservoir at Fort Riley, Kansas, U.S.A.[1]—The capacity of

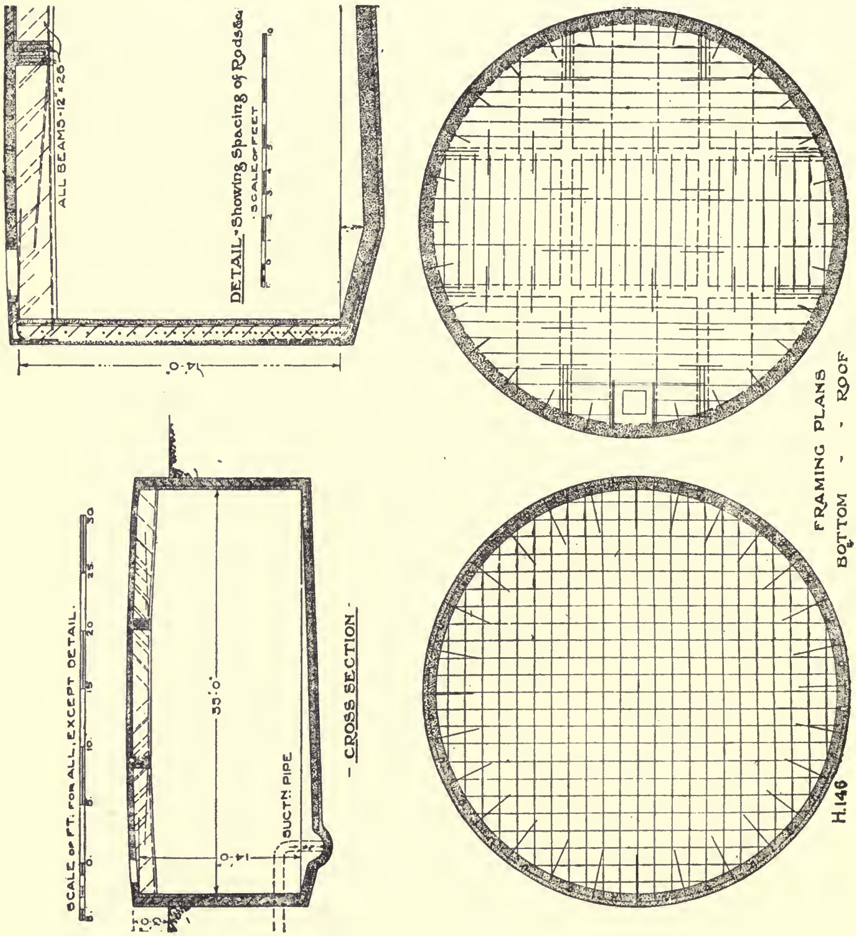


Fig. 130.

"Concrete and Constructional Engineering."

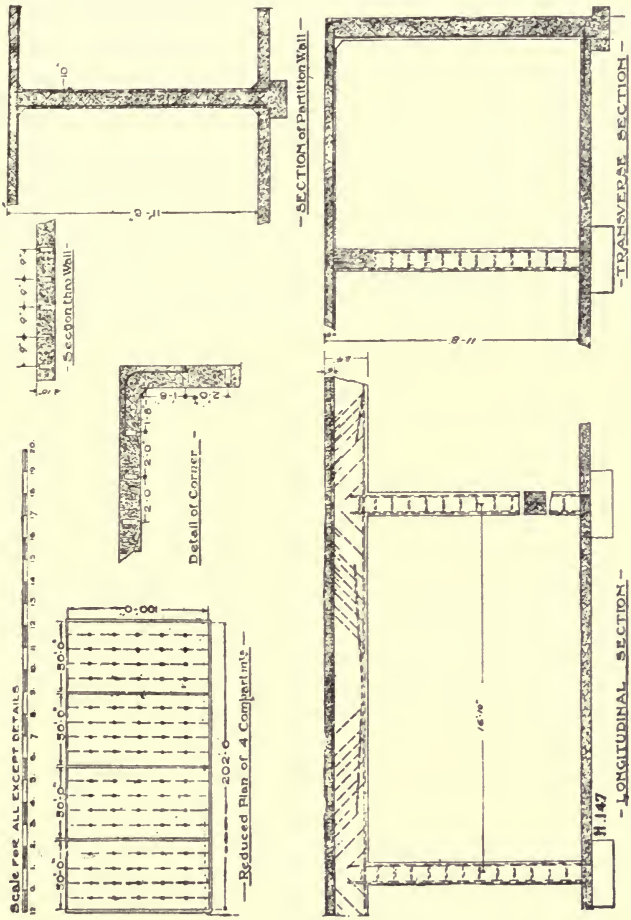
this reservoir is 2,000,000 gallons. It is reinforced on the Kahn system.

Reservoir at Pudsey, Yorkshire.—An open reservoir has

[1] "Reinforced Concrete Reservoirs," by E. R. Matthews. "Concrete and Constructional Engineering," July, 1909, p. 163.

recently been constructed at Pudsey for Messrs. W. C. Forrest & Co., of Prospect Mill, the engineers being Messrs. Jowett, Kendall & Son of Pudsey.

The reservoir is constructed throughout of reinforced con-



"Concrete and Constructional Engineering."

Fig. 131.

crete, the walls being only 8 in. in thickness although they stand up 9 ft. above ground level. They are, however, strengthened by counterforts. The capacity of the reservoir is 500,000 gallons, and the construction is clearly shown in Fig.

133, and in the photograph (Fig. 132). Near the top of the wall is a band to stiffen the structure.

Fig. 132.

The arrangement of the reinforcement is shown in Fig. 133. The work has been carried out by the Chain Concrete Syndicate of Leeds, upon their patent system of reinforcement, the chief feature of which is their connecting clips and stirrups; these are illustrated in the following diagrams (Figs. 134 and 135).

This section dealing with reservoirs and tanks would not be complete were no reference to be made to the excellent examples of work of this description done by the Hennebique Contracting Co., Ltd., who are one of the pioneer firms in reinforced concrete construction in this country. Their system consists in the introduction of a series of special stirrups (see diagram), also straight and bent bars arranged in a particular manner.

Fig. 136 is a diagram showing the respective positions of the bars and stirrups, while Fig. 137 shows the arrangement of the reinforcement in columns and beams.

Water Tank at Trafalgar Mills, Huddersfield.—This is represented in Fig. 138. The tank has a capacity of 209,000 gallons. Its dimensions are 103 ft. by 60 ft. by 6 ft. deep, and it is divided into three compartments. The average thickness of the walls and bottom is 5 in.

Other illustrations of tanks constructed by this firm are described under the heading of "Water Towers".

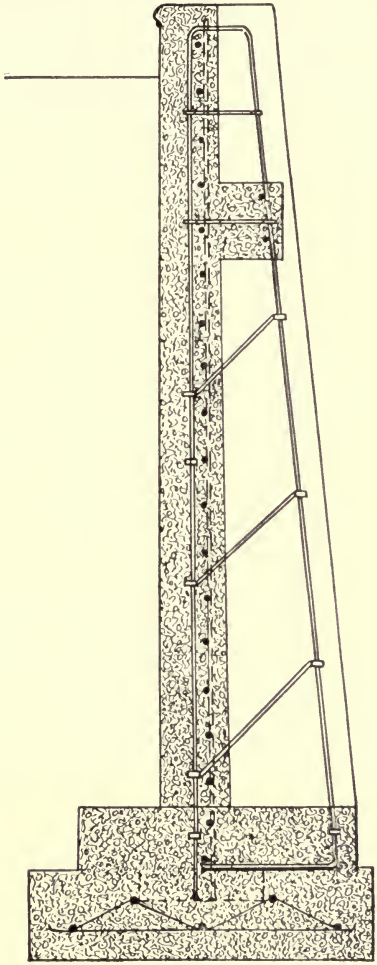


FIG. 133.

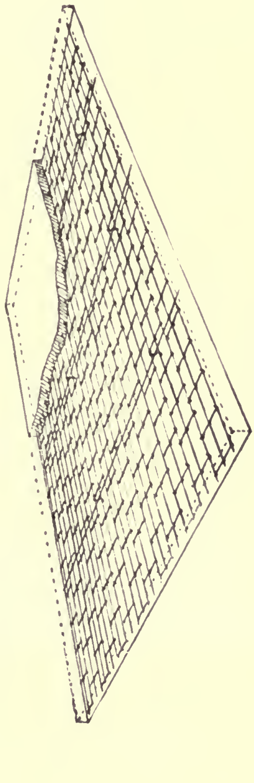


FIG. 134.

WATER TOWERS.

The material is undoubtedly eminently suitable for the construction of water towers, for which purpose it is being largely used. The authors will briefly describe a few works of this class which have recently been carried out in this country.

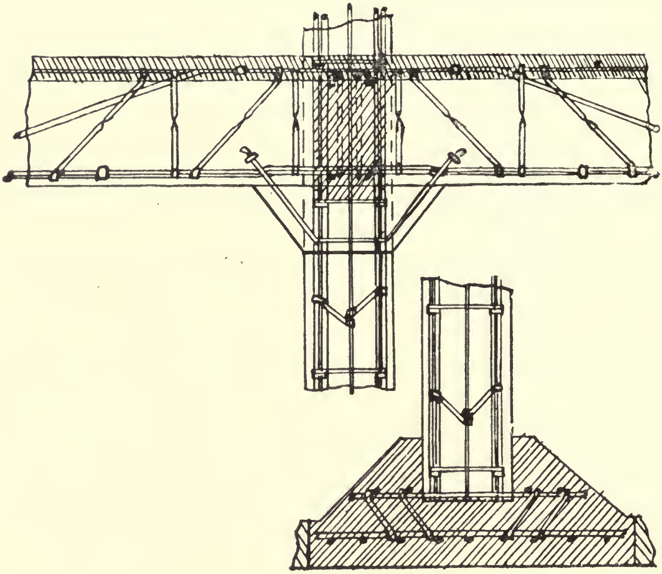


FIG. 135.

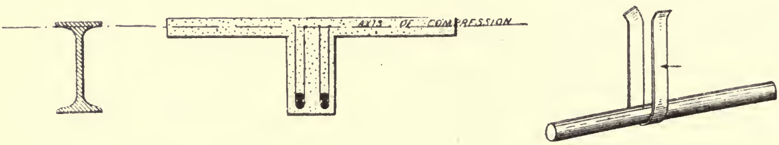


FIG. 136.

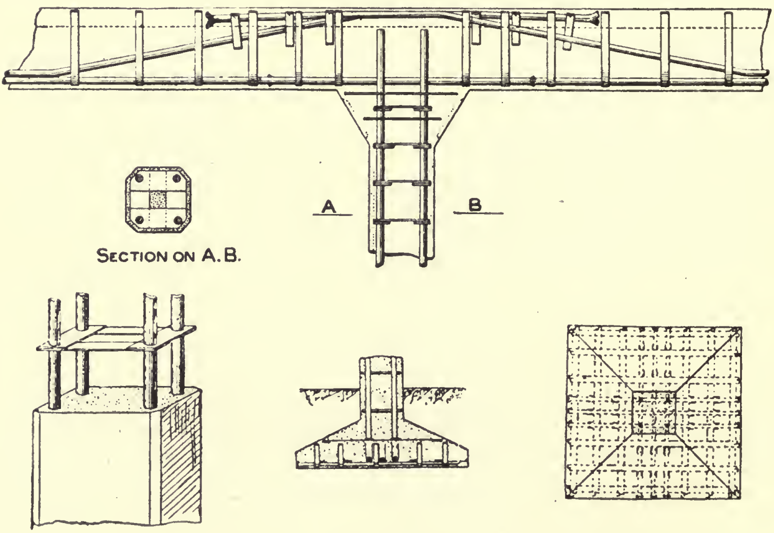


FIG. 137.

Rhyl Water Tower.—This tower (see Fig. 139) designed by the Patent Indented Steel Bar Co., has been erected for the Rhyl Town Council. It comprises a circular tank 31 ft. internal diameter and 12 ft. deep with a capacity for storing 50,000 gallons. The tank is supported by octagonal columns, which are stiffened by three tiers of lateral wind bracing.

The maximum height of the tower is 73 ft. 6 in. above ground level, and the thickness of the walls and floor of the tank is 6 in.

Fig. 138.

The tank has been erected in a very exposed position near the sea, but has successfully resisted the wind stresses. No rendering or waterproof coating has been applied to the tank; but it is nevertheless perfectly watertight.

The aggregate used consisted of sand and broken Welsh granite 1 : 2 : 4.

Tower at Cleethorpes.—This is shown in Fig. 140, and is a very good example of a reinforced concrete water tower. It is one of the highest tanks in existence, being 174 ft. above ground level. Its storage capacity is 250,000 gallons. It has been erected for the Great Grimsby Waterworks Co., Ltd., under the direction of Mr. H. Hewins, the chief engineer to that Com-

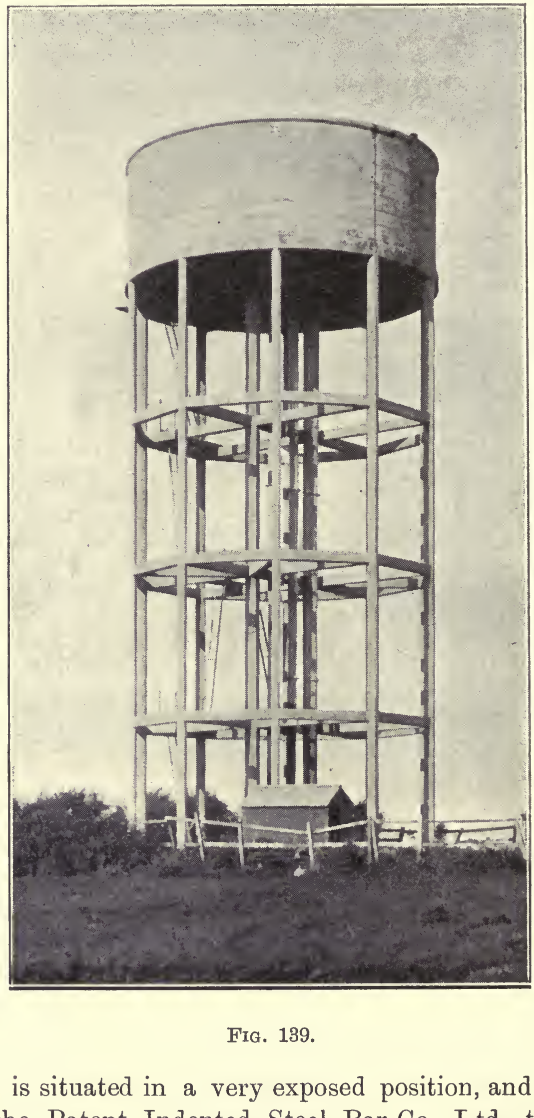


Fig. 139.

pany. It is situated in a very exposed position, and has been built by the Patent Indented Steel Bar Co., Ltd., the special feature of whose system consists in the patent indented bars that are used, which they claim give an excellent mechanical bond, which is not obtained by the use of plain bars (see diagram).

FIG. 140.

The diameter of this tower is 40 ft., the water tank is of steel, and is 42 ft. high and 34 ft. in diameter; it is supported on a special ring girder which provides for expansion.

Tower at Immingham.—This is illustrated in Fig. 141, and is a combination of the two types before mentioned, for although the supports of the tank are in the form of columns, yet they

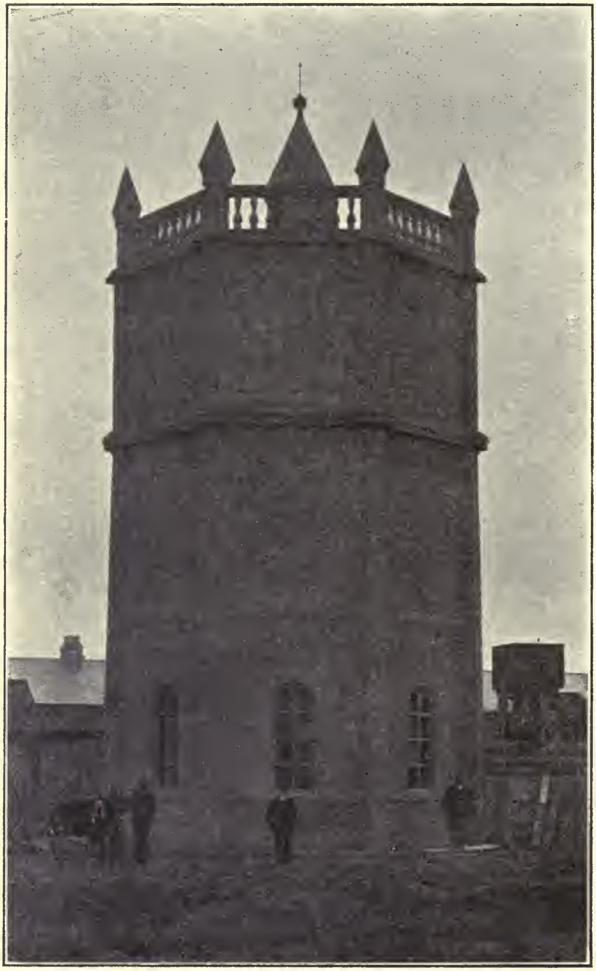


FIG. 141.

are spaced around the perimeter of the tank, and reinforced concrete walls are constructed between the columns. The tower is octagonal in shape, and the height above ground of the water level is 45 ft.; the width from side to side is 26 ft.; the depth of the tank is 12 ft. 6 in. and its storage capacity 40,000 gallons.

With a view to a further extension of 40 ft. the columns are continued above the roof of tank.

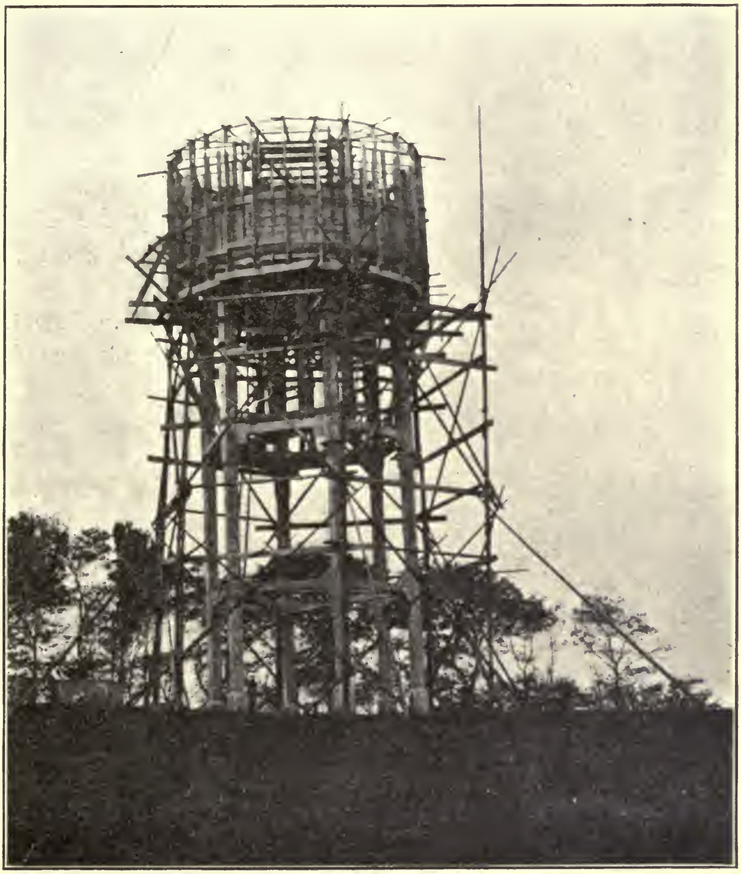
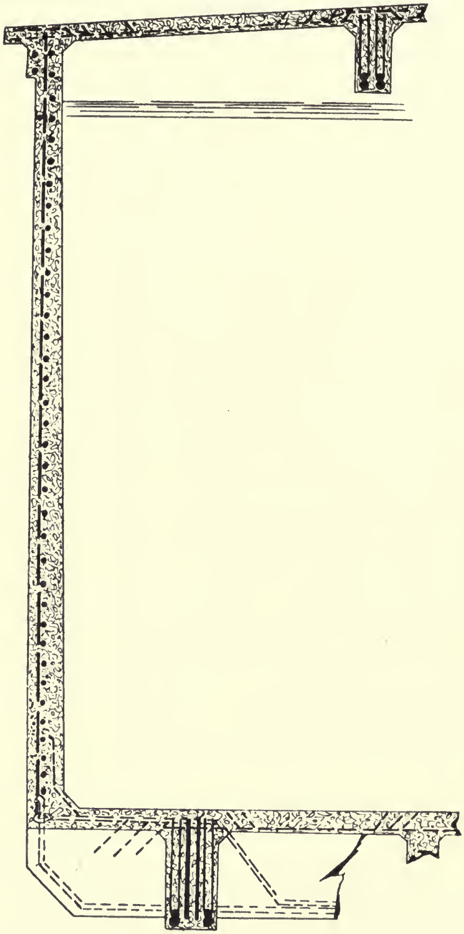


FIG. 142.

The design was prepared, and the work carried out under the supervision of Mr. Henry Hewins, engineer to the Great Grimsby Waterworks Co. The walls of the tank are 8 in. in thickness at the bottom, and $4\frac{1}{2}$ in. at the top. The bottom of the tank assumes the shape of an inverted pyramid, and is



6 in. thick. The walls of the tank are strengthened by the addition of horizontal and vertical ribs. The depth of water in the tank is 30 ft. notwithstanding which it is absolutely water-tight. The work was carried out by the Patent Indented Steel Bar Co.

Dudley Tower.—This is illustrated in Fig. 142. It has been built for the South Staffordshire Waterworks Co., also by the same firm. It has a capacity of 20,000 gallons, is 51 ft. in height, and has been erected on a reinforced concrete raft owing to the unsatisfactory nature of the ground.

Covered Water Storage Tank.—The British Reinforced Concrete Engineering Co. are now erecting the tower illustrated in Fig. 143. The tank is circular in shape, being 26 ft.

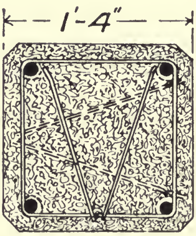


FIG. 143.

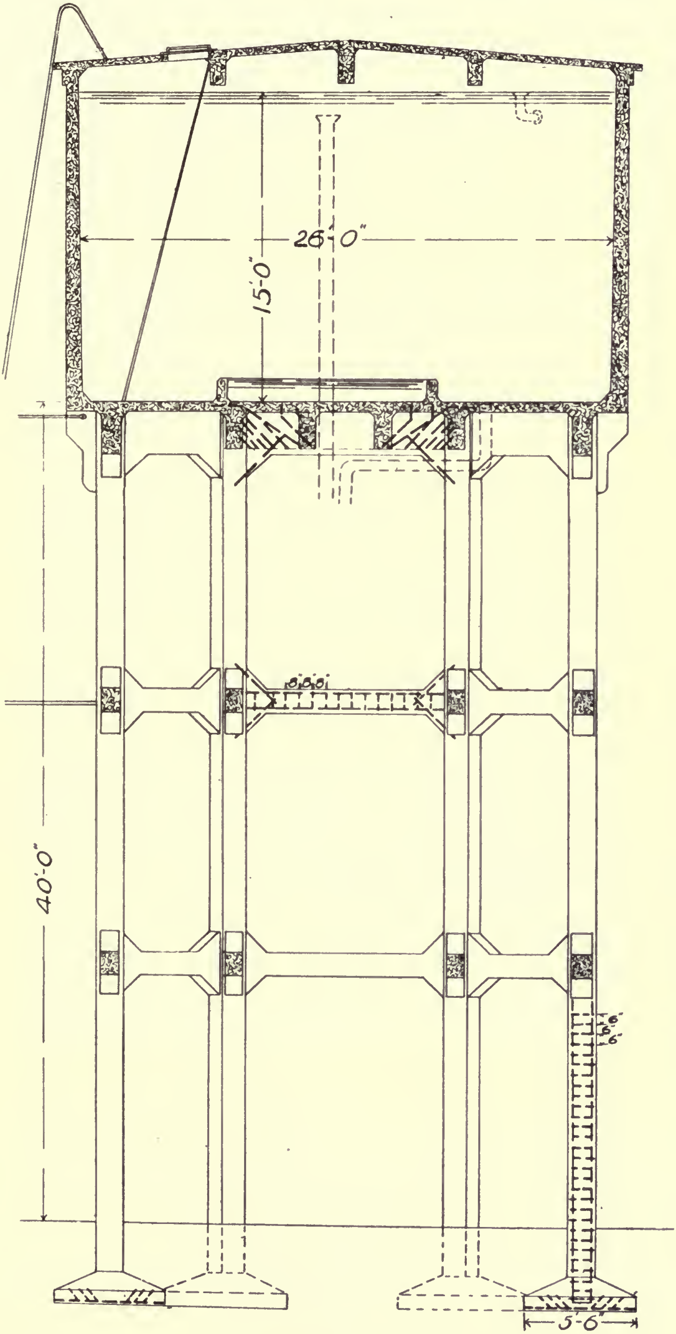


FIG. 143A.

internal diameter, and allows for a maximum depth of water of 15 ft. It is carried by reinforced concrete beams which rest on reinforced concrete square piers or columns 1 ft. 4 in. by 1 ft. 4 in. The arrangement of the reinforcement, and position of the braces are shown in the illustration. The tank is 40 ft. above ground level.

Fig. 144.

The Hennebique Contracting Co. have erected a number of water towers, a brief description of five of them being given :—

Water Tower at Newby Hall, near Ripon.—This is shown in Fig. 144. It has a capacity of 40,000 gallons, is 24 ft. in diameter, 16 ft. deep, and the bottom of tank is 65 ft. above ground level. It is erected on a continuous slab foundation.

Water Tower at Barmby Moor, near Pocklington.—This has a capacity of 41,000 gallons. It is constructed throughout of reinforced concrete, is 27 ft. diameter, 12 ft. 6 in. deep, and the height to top of tank is 34 ft. 3 in. (see Fig. 145).

Fig. 145.

Tower at Milford Junction, Yorkshire.—This tower has been erected for the N.E. Railway Co. Its capacity is 20,000 gallons, diameter 22 ft. 6 in., depth of water 9 ft. ; height above ground level over all 37 ft. It has been built on a reinforced concrete pile foundation (see Fig. 146).

Tower at Gascoigne Wood for N.E. Railway Co.—This tank (see Fig. 147) is 34 ft. internal diameter, 12 ft. deep, and the bottom of tank is 60 ft. above ground level. The structure has been built on a pile foundation.

Water Tower at Londesborough, near York.—This has been built for the Pocklington Rural District Council. The tank has

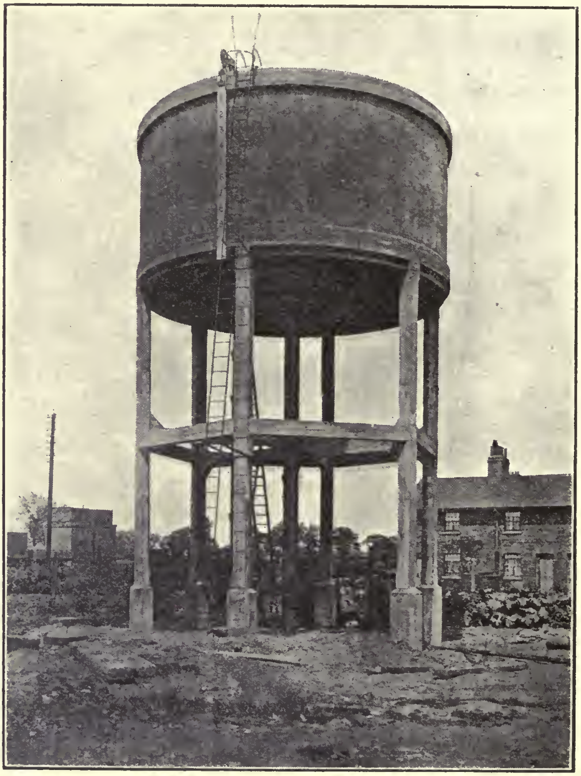


Fig. 146.

a capacity of 20,000 gallons, and the height of the top of same above ground is 34 ft. (see Fig. 148).

Tower at Down District Asylum, Downpatrick.—This interesting water tower, shown in Fig. 149, was designed by Mr. James Heron, County Surveyor, the contractor being Mr. R. D. Pollock. The tank, which is 16 ft. outside diameter and 11 ft. in depth, is supported by a tower 13 ft. 4 in. in diameter. The walls of the tower are 8 in. in thickness, and those of the tank 6 in. The roof slab of tank is also 6 in. thick, and the floor 12 in. There is storage capacity for a depth of water of 10 ft., and the total height of the structure, above ground level, is 55 ft. The reinforcement is expanded metal.

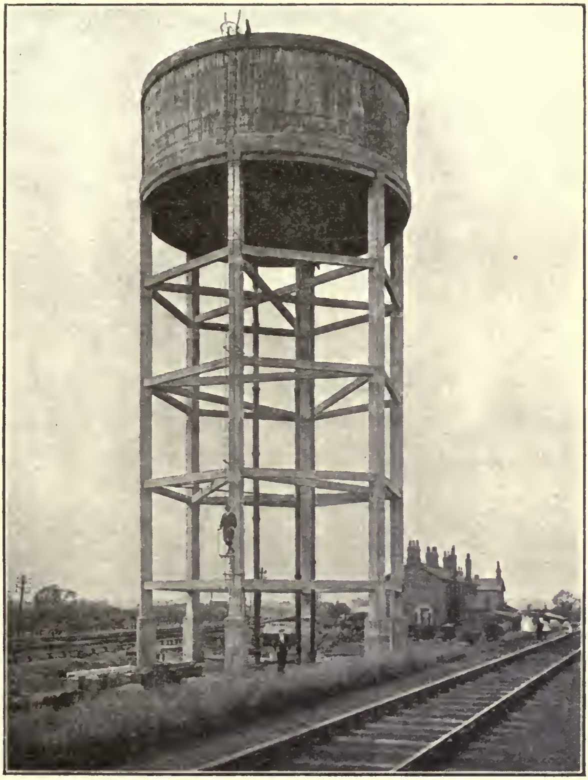


Fig. 147.

Culvert and Conduit Construction.—Most of the advantages claimed for reinforced concrete in connection with reservoir construction, are also applicable to its use in the construction of culverts, aqueducts, and conduits. On the Continent and in America this material has been used largely for this purpose.[1]

[1] "Reinforced Concrete Sewers and Conduits," by E. R. Matthews, Vol. CLXVII (1906-1907), "Proc. Inst. C.E."; "Reinforced Concrete Sewer Con-

Two excellent examples might be referred to.

The conduit constructed in 1903 in connection with the water supply of Jersey City, U.S.A., and the new sewers at San Francisco. Fig. 150 is a typical cross-section through the former, and the photo illustrates the reinforcement (indented bars) in con-

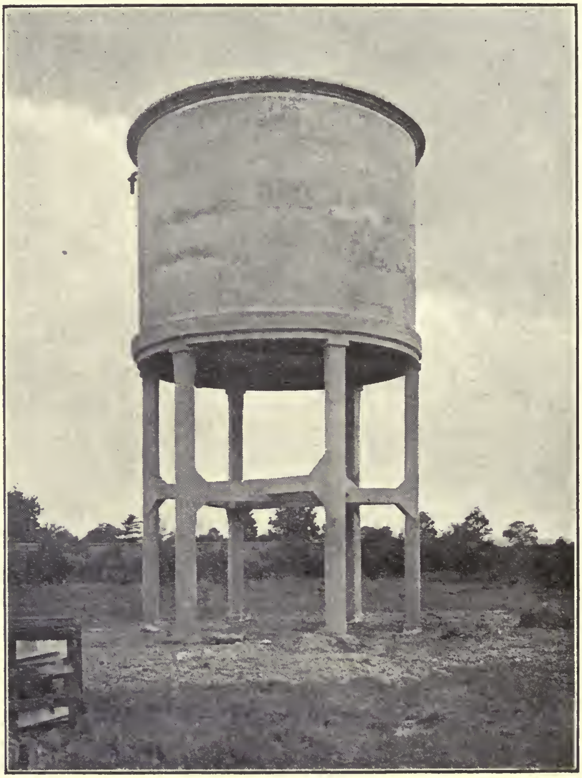


Fig. 148.

nection with the 6 ft. outfall sewer in Mariposa Street, San Francisco; both represent excellent examples of the use of reinforced concrete in conduit construction.

As to English examples of the use of this material for this

struction in San Francisco," by E. R. Matthews, "Concrete and Constructional Engineering".

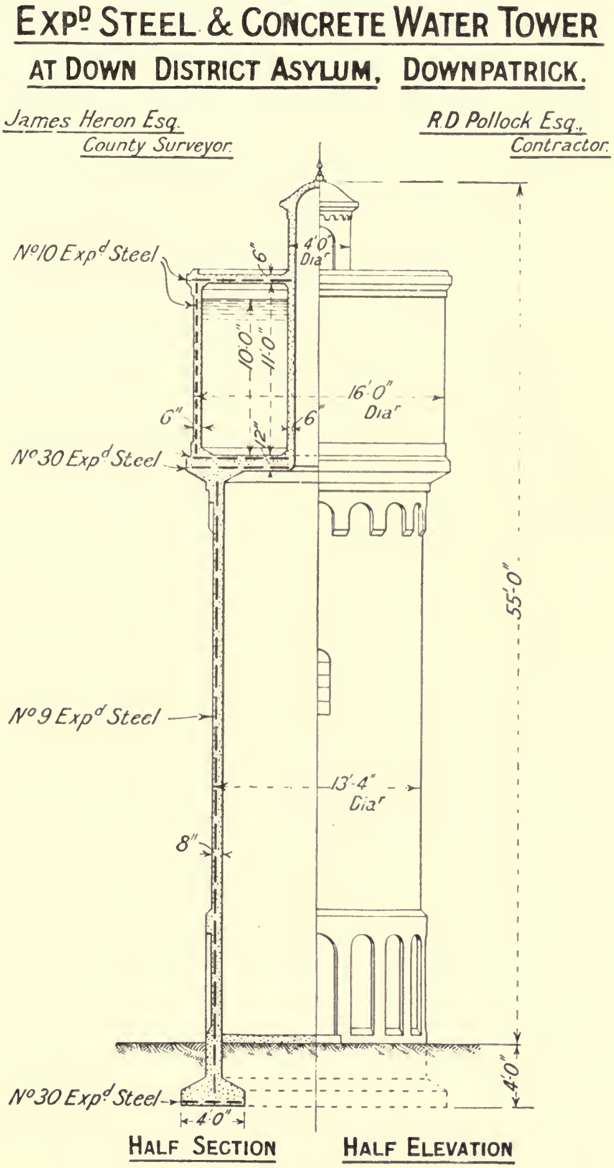

EXPD STEEL & CONCRETE WATER TOWER
AT DOWN DISTRICT ASYLUM, DOWNPATRICK.
James Heron Esq. County Surveyor.
R.D Pollock Esq., Contractor.
No10 Expd Steel
4'0" Diar
6"
10'0"
11'0"
16'0" Diar
6"
6"
12"
No30 Expd Steel
55'0"
No9 Expd Steel
13'-4" Diar
8"
No30 Expd Steel
4'0"
4'0"
HALF SECTION
HALF ELEVATION

FIG. 149.

purpose, many might be given; the authors, however, give one or two typical examples.

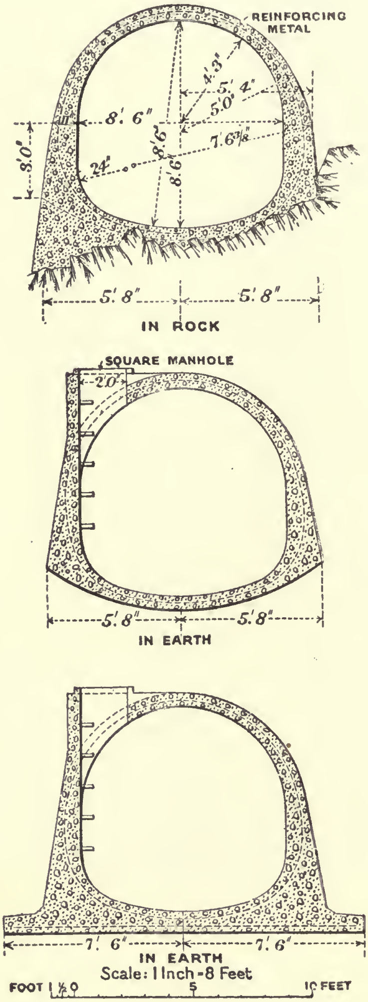


FIG. 150.

Culvert at Bromboro' Port.—Fig. 153 represents a large culvert erected by the British Reinforced Concrete Engineering Co., at Bromboro' Port. It is a good illustration of what can

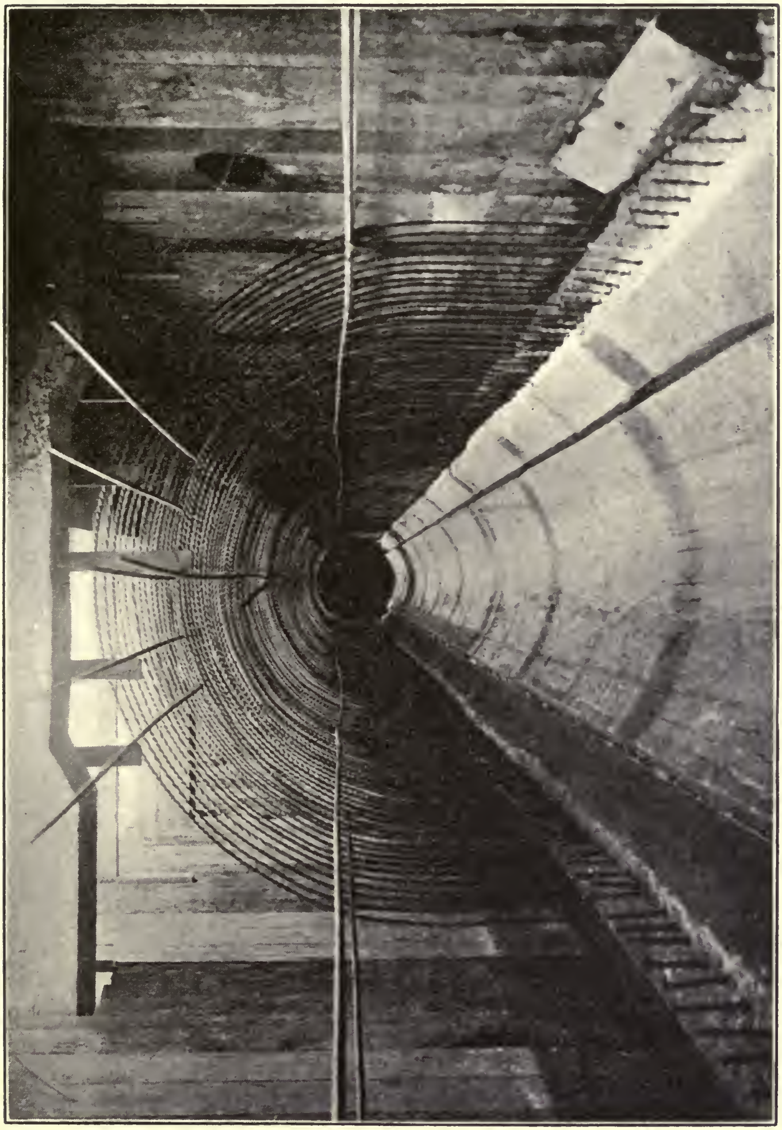
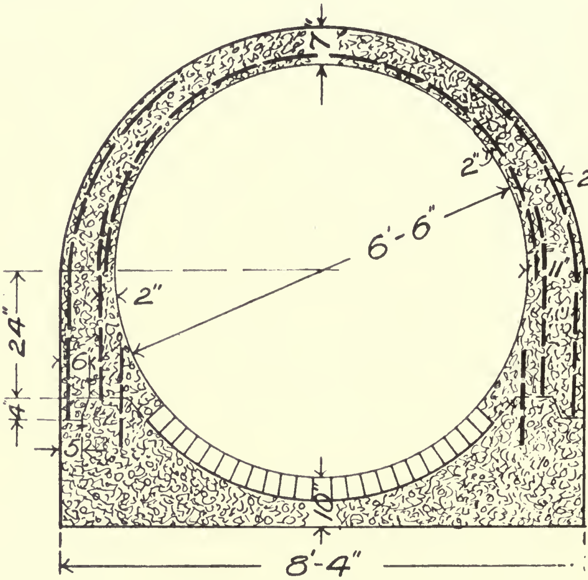


Fig. 151.

"Concrete and Constructional Engineering."

be done in this material. Messrs. William and Segar Owen were the engineers.



"*Concrete and Constructional Engineering.*"

FIG. 152.

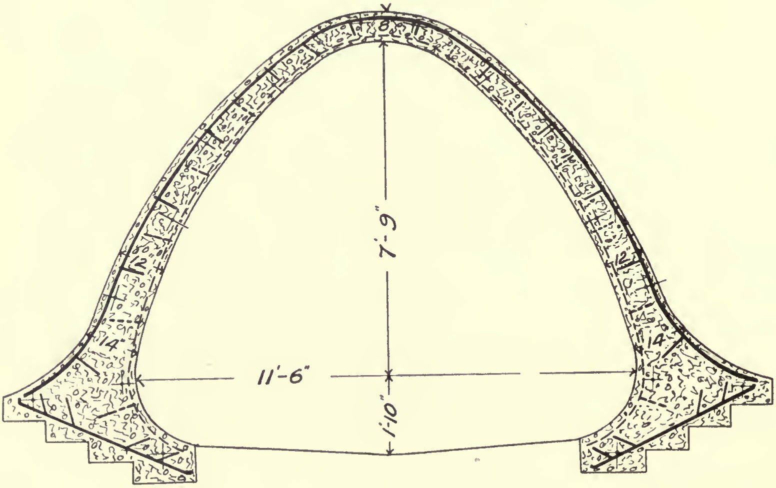


FIG. 153.

Blaenavon Culvert.—This is shown in Fig. 154. It has been

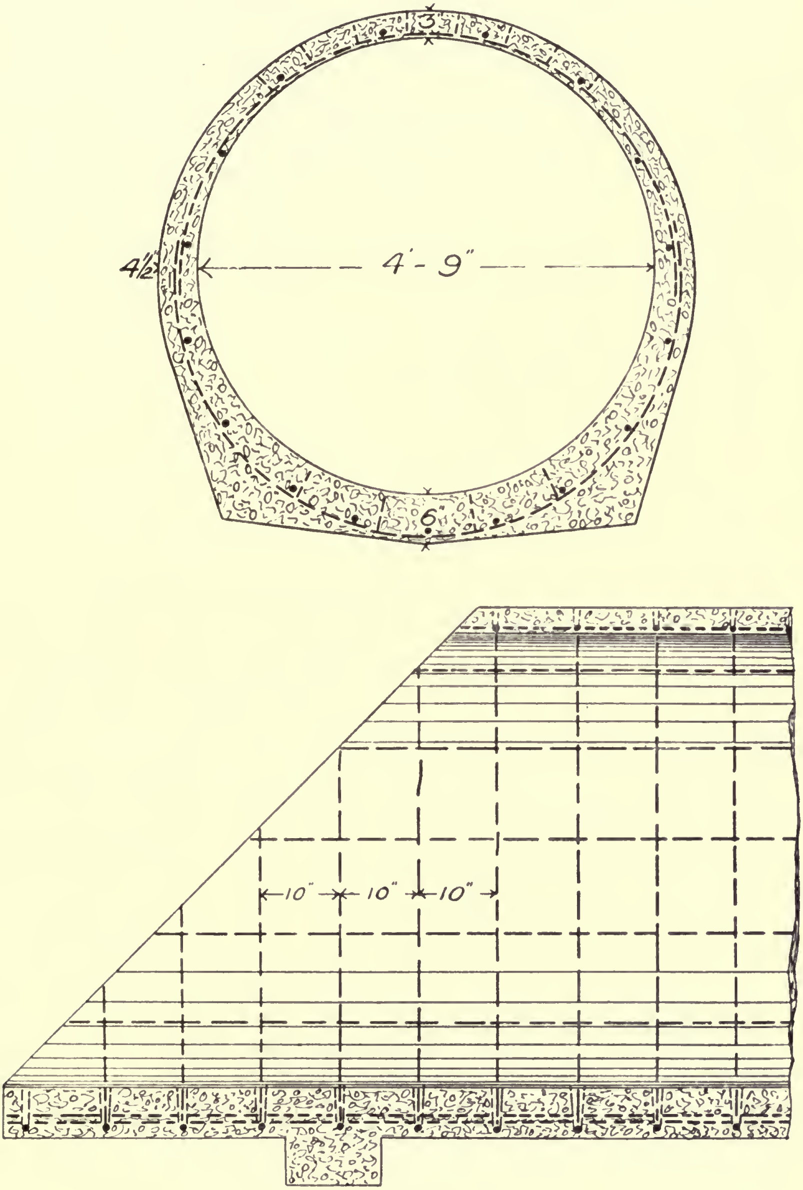


FIG. 154.

constructed by the same firm for the Blaenavon Urban District Council, South Wales; it is 4 ft. 9 in. in diameter, and the thickness varies from 6 to 3 in. It has a square base, and is reinforced in the manner shown.

Kilton Culvert for N.E.R. Co.—This is an important culvert constructed on the Kahn system by the Trussed Concrete Steel Co., and designed by the late Mr. W. J. Cudworth, M.Inst.C.E. It is 446 ft. in length, and of the section shown in Fig. 155. The photos Figs. 156 and 157 also illustrate the culvert. The work was completed in 1908. Fig. 158 shows the timbering, and Fig. 157 the test load being applied. The aggregate used consisted of 3 parts of a mixture of gravel and crushed whinstone, $\frac{3}{4}$ in. gauge, with sufficient sand to fill interstices, to 1 of cement.

Expanded metal has been used very considerably in conduit construction, Figs. 159 to 163 representing its use in a variety of such ways.

The Lock Woven Mesh System has also been adopted in many cases, Fig. 164 representing an 8 ft. Penstock 4600 ft. long, which is carried out on this system.

Concrete Pipes.

In America these are used very considerably, sometimes plain, but generally reinforced. For water mains the Bonna pipe is chiefly used: this is too well known to need description. Bonna pipes have recently been used for the construction of a rising main for the Norwich City Council, Mr. A. E. Collins, M.Inst.C.E., being the engineer. This new sewage pumping main is 36 in. internal diameter, and about 4500 yards in length. The work has been executed by the Columbian Fireproofing Co., the cost of the pipes including specials being upwards of £9000. Each pipe weighs about $1\frac{1}{2}$ tons; the work has been well tested, and is in every way satisfactory[1] (see Fig. 166).

On the Wabash Railroad, U.S.A., reinforced concrete pipes have been made of sections varying from 2 to 4 ft.[2]

[1] "Reinforced Concrete Rising Main on the Bonna System at Norwich," by E. R. Matthews, "Concrete and Constructional Engineering," December, 1910.

[2] "Reinforced Concrete on the Wabash Railroad, U.S.A.," by Matthews and Cunningham (1909-1910).

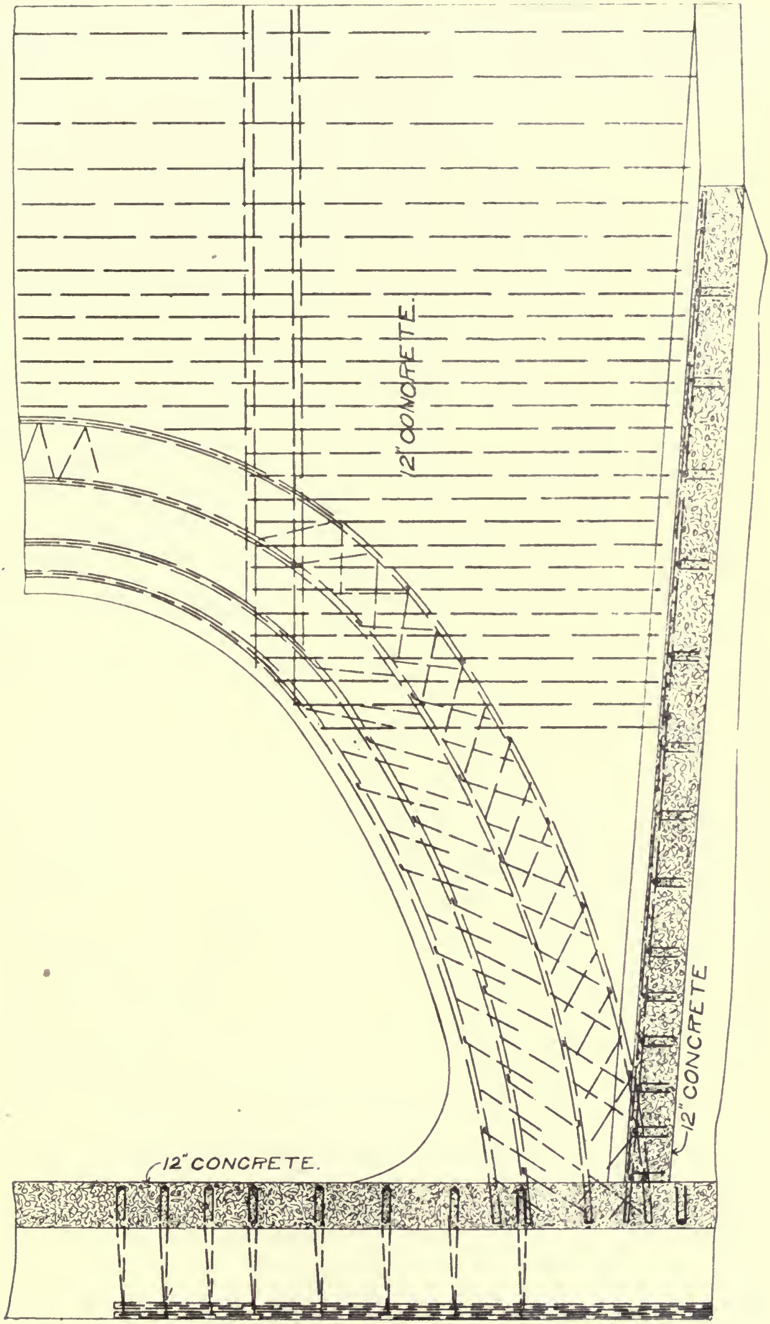


FIG. 155.

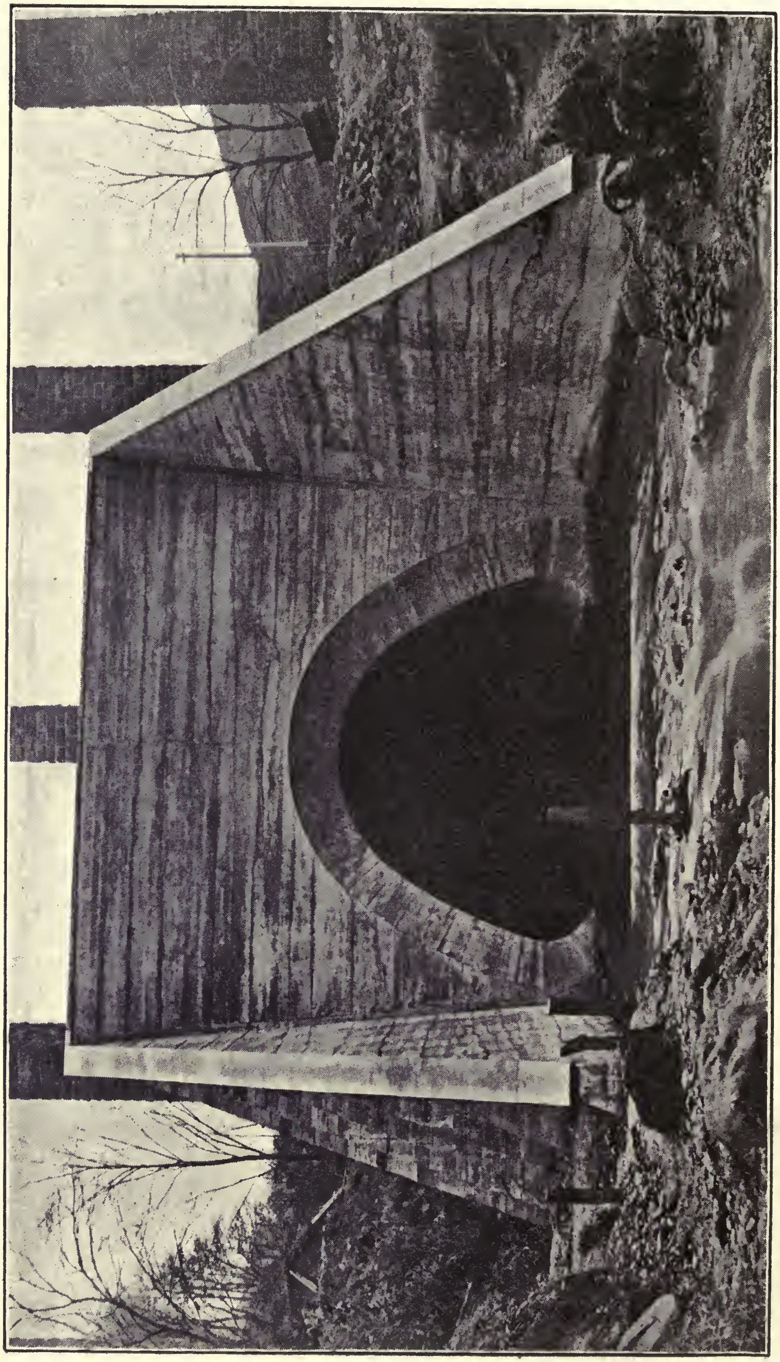


Fig. 156.

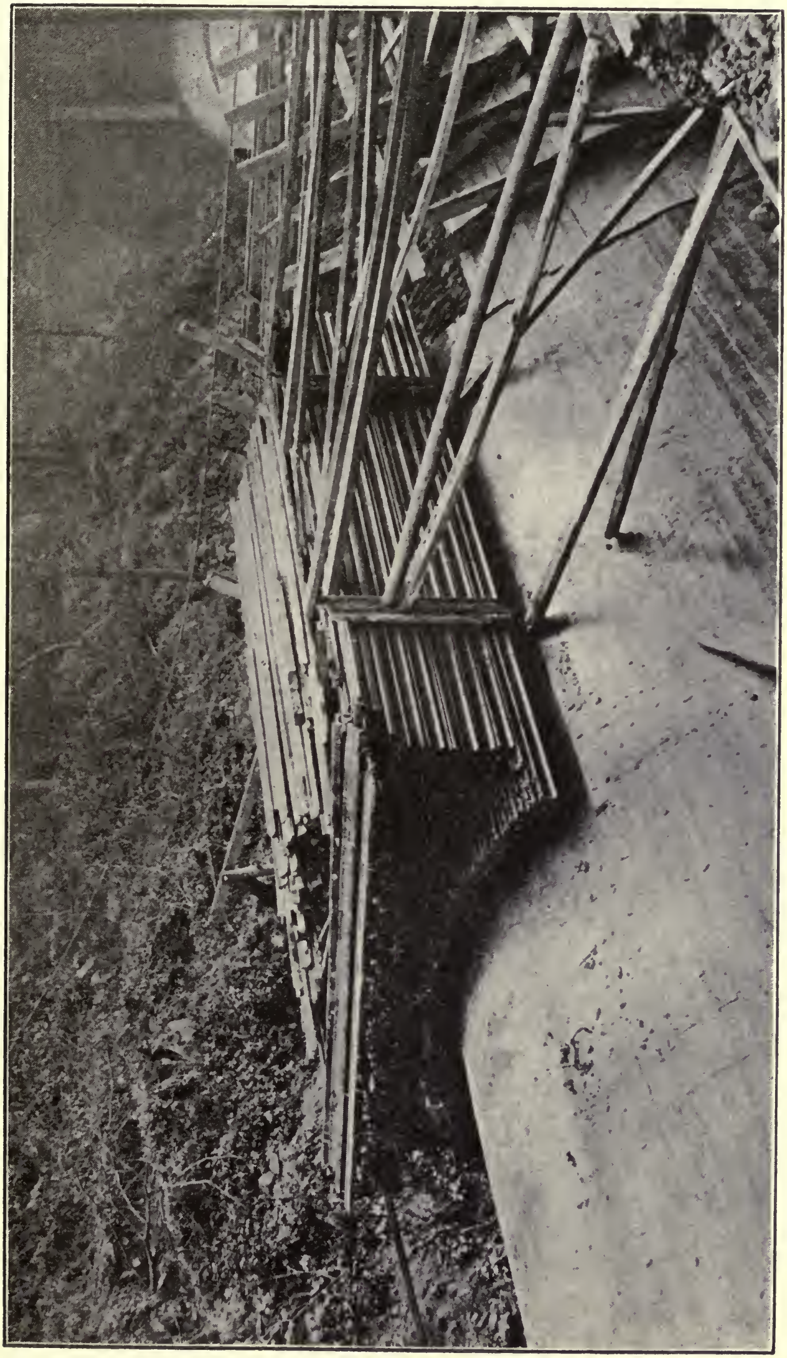


Fig. 157.

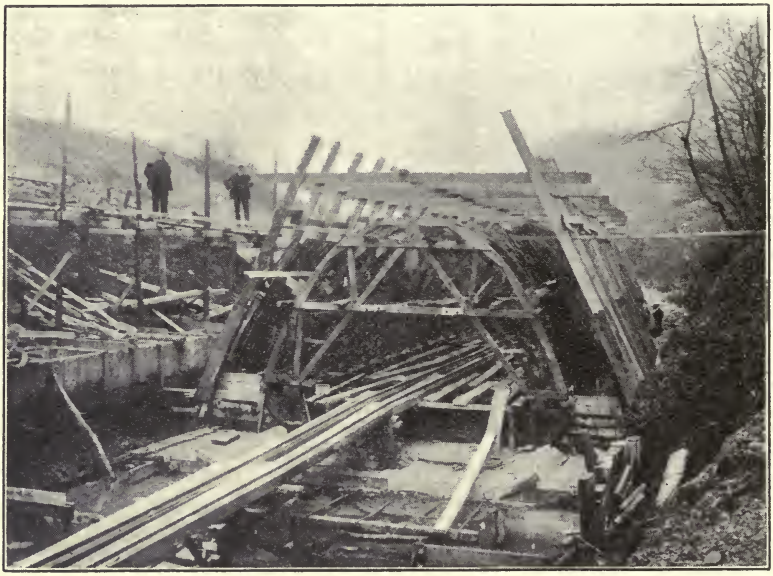


Fig. 158.

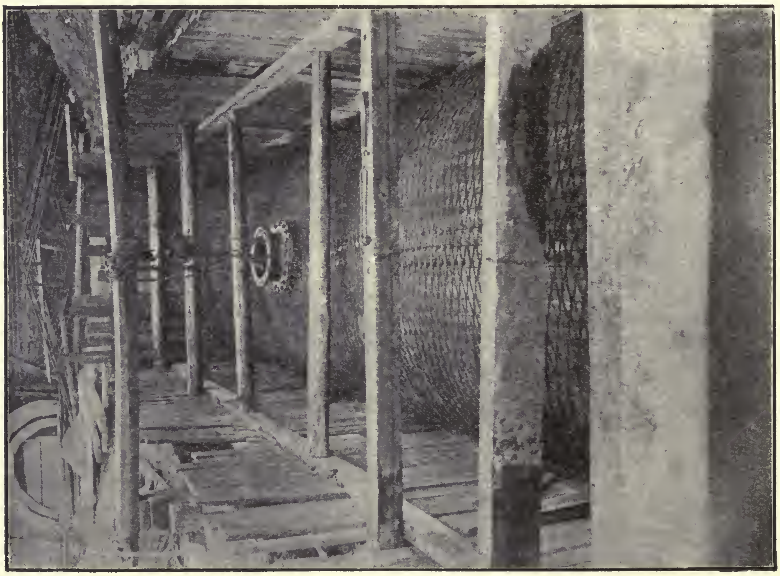


Fig. 159.

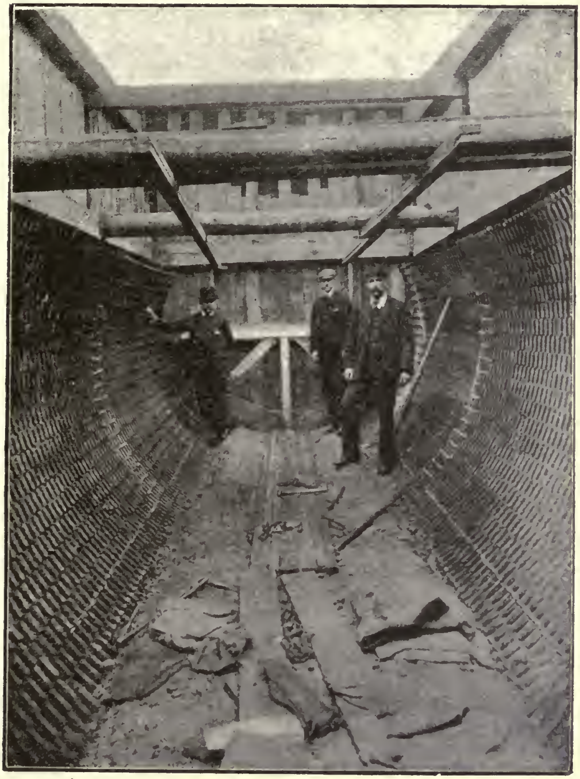


FIG. 160.

EXPANDED STEEL & CONCRETE SEWER
FOR THE CALICO PRINTERS ASSOCIATION LTD.
at Reddish Vale, near Stockport.

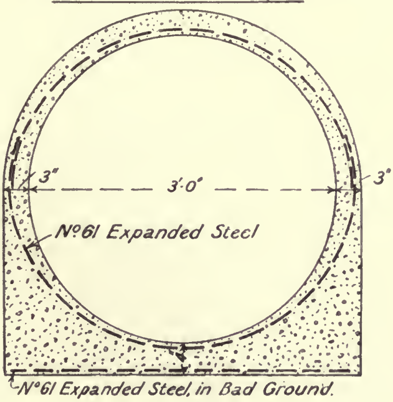


FIG. 161.

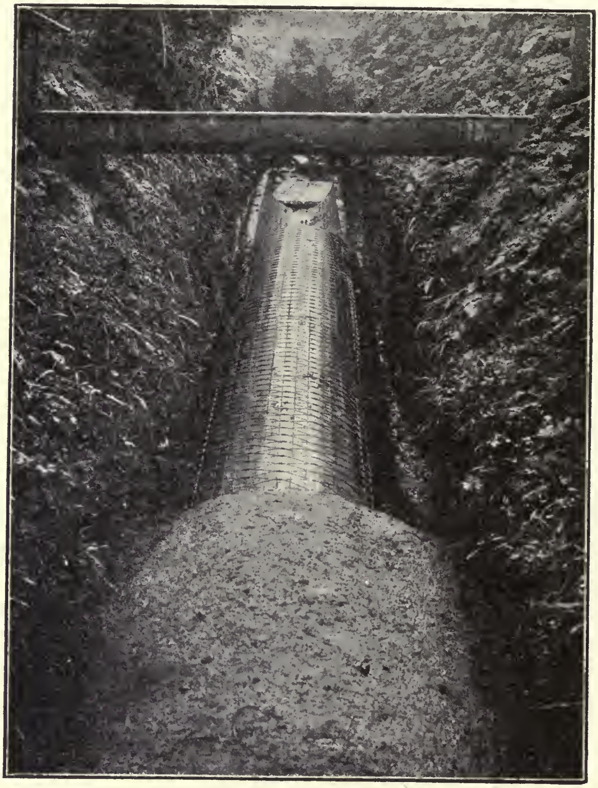


Fig. 162.

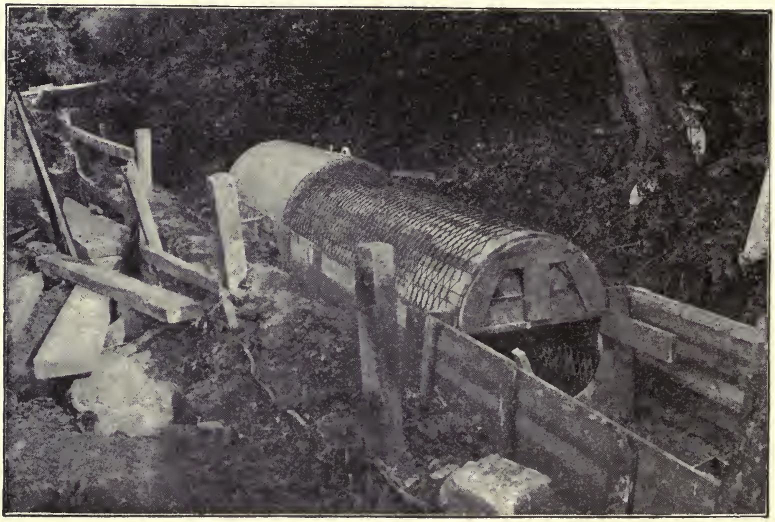


Fig. 163.

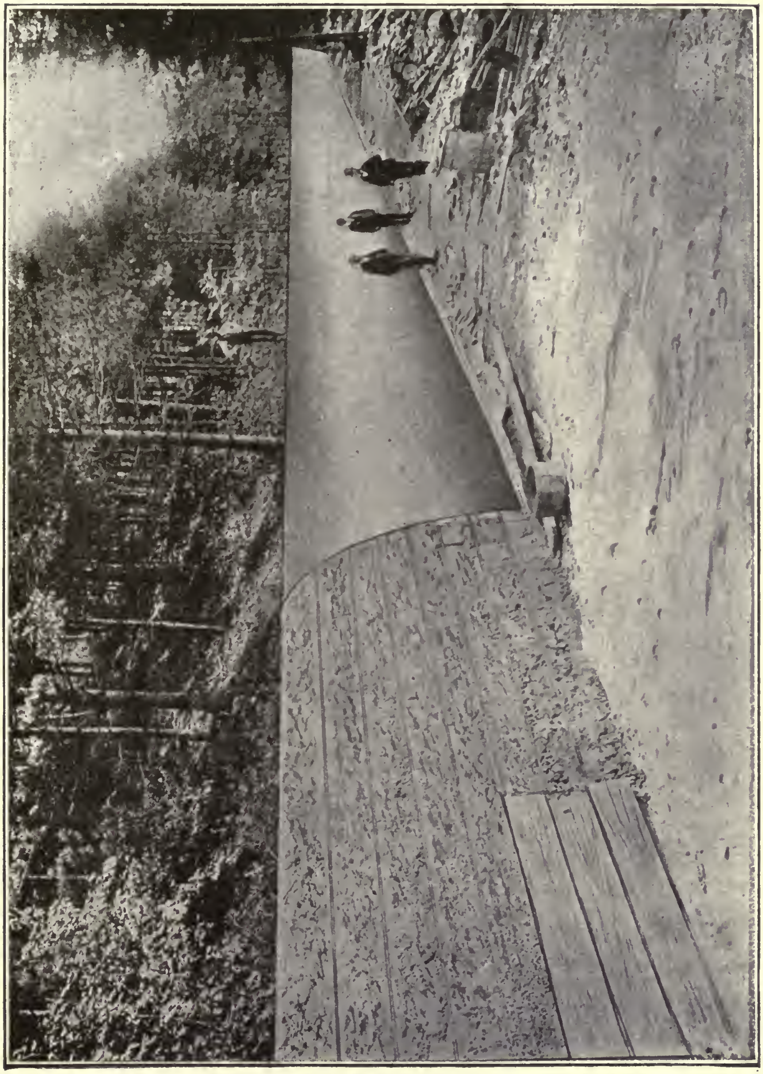


Fig. 161.

Fig. 165.

"Concrete and Constructional Engineering."
FIG. 166.

Fig. 167 illustrates a 4 ft. type. The reinforcement consists of woven-wire fencing.

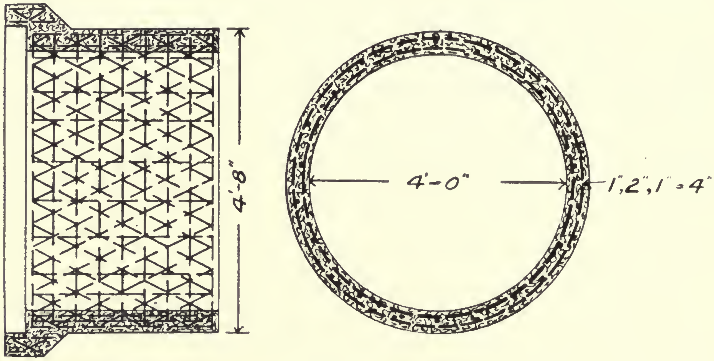


FIG. 167.

The quantity of material required to make a 4 ft. pipe, 3 ft. in length is :—

Portland cement	1 barrel = 376 lb. = 3·8 cub. ft.
Sand	0·3 cub. yd.
Stone or gravel (capable of passing through ½ in. ring) . . .	0·61 cub. yd.
Wire fencing 34 in. wide . . .	28 lin. ft.
½ in. corrugated steel bars (3 ft. long)	22 lin. ft.

A number of English firms now manufacture reinforced concrete pipes, one of the best-known firms being Messrs. Ellis & Sons. Figs. 168 and 169 illustrate some of the pipes manufactured by this firm.

SEWAGE TANKS.

Tank at Ripponden.—The material has also been used very largely in the construction of sewage disposal works. Fig. 170 represents a sewage tank at Ripponden, designed by Mr. F. Gordon, engineer to the Soyland Urban District Council. The work has been executed by the Yorkshire Hennebique Contracting Co.

Reinforced Concrete Sewage Disposal Works. — Sewage disposal works constructed by the British Reinforced Concrete


F. G. BRUMMELL.
Road & Sewer Contractor
Yard 1 Dudding Hill Lane
Willesden Green.

FIG. 168.

Fig. 169.

Engineering Co. are shown in Fig. 171. It will be noticed that there is a pair of liquefying tanks and filters, the former being covered.

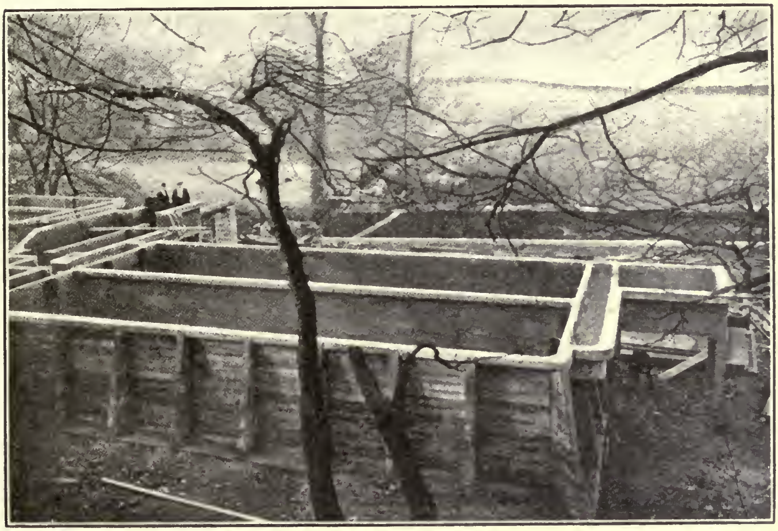


Fig. 170.

Swimming Baths.

Many excellent works of this class might be referred to; the authors will draw the attention of the reader however to two only, which may be taken as typical examples.

Salford Corporation Public Baths.—These are represented in Fig. 172. They were designed by Messrs. Mangnall & Littlewoods, architects, Manchester, the work being carried out by the Fram Fireproof Construction Co. The reinforcement is expanded metal, arranged as shown in the section. The dimensions of these baths are: length 75 ft., width 30 ft., depth 4 ft. 4 in. to 7 ft. 4 in.

The ladies baths, also of reinforced concrete, are somewhat smaller, being 50 ft. by 25 ft. The walls in both baths are buttressed.

South Shields Public Baths.—These are shown in Fig. 173. They are constructed throughout of reinforced concrete, and

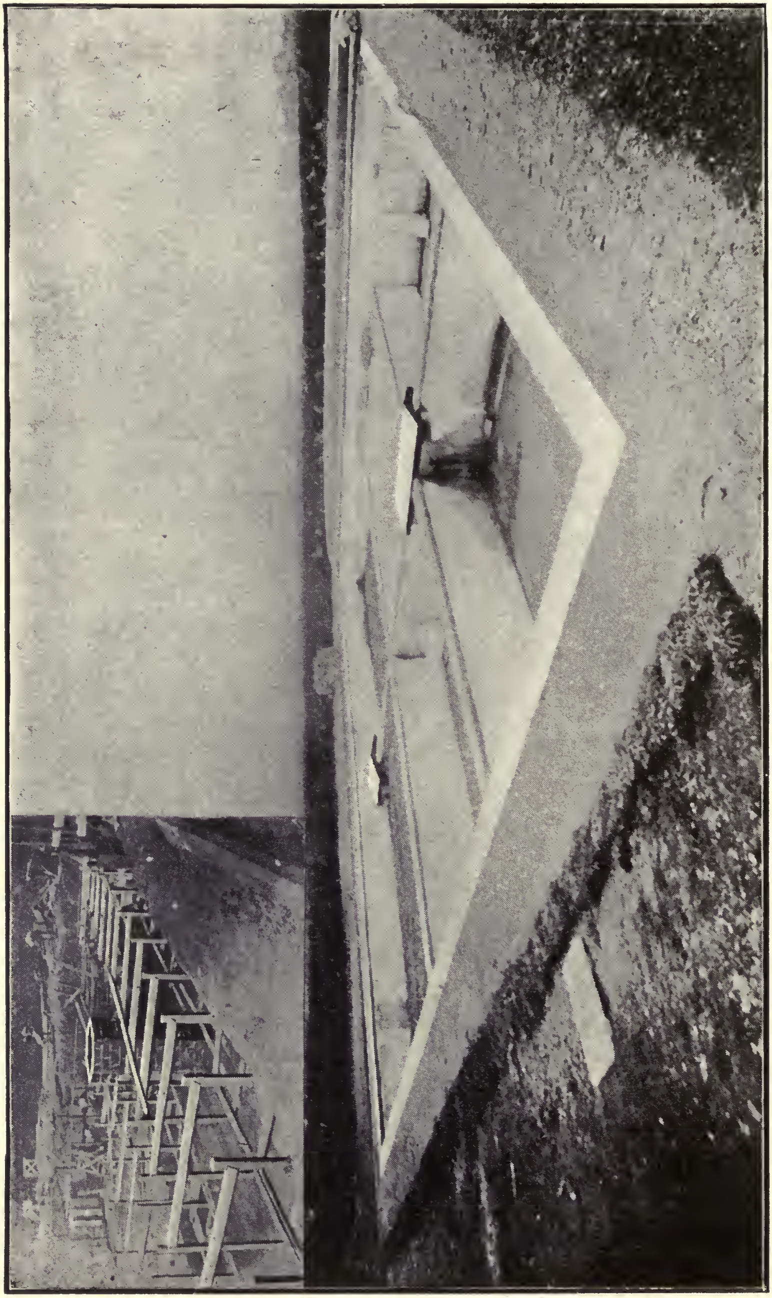


Fig. 171.

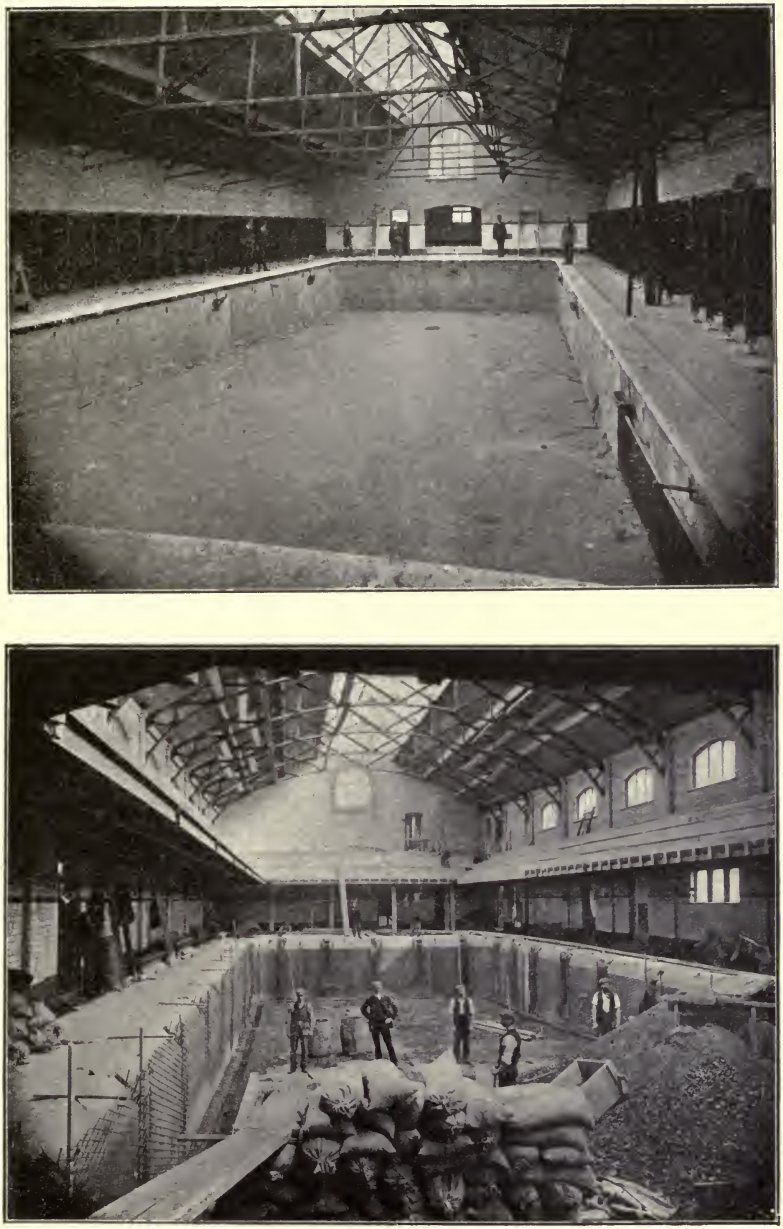


Fig. 172.

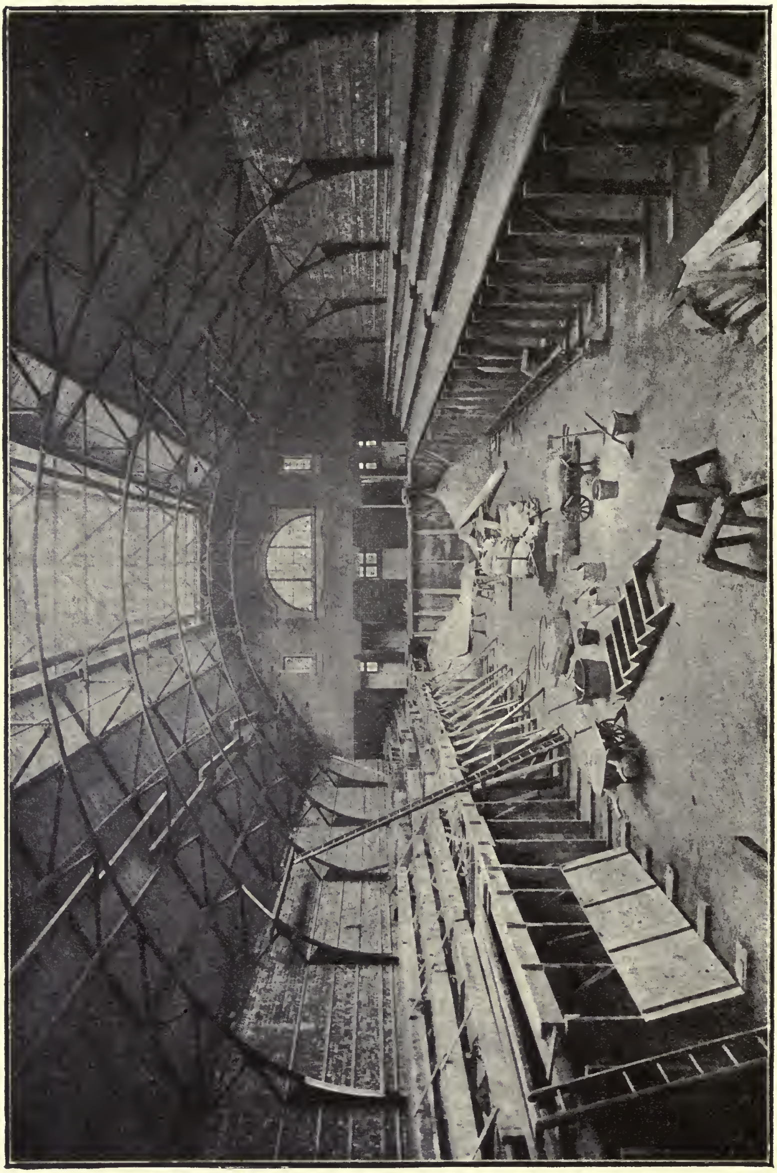


FIG. 173.

were designed by Mr. J. H. Morton, F.R.I.B.A., architect of South Shields.

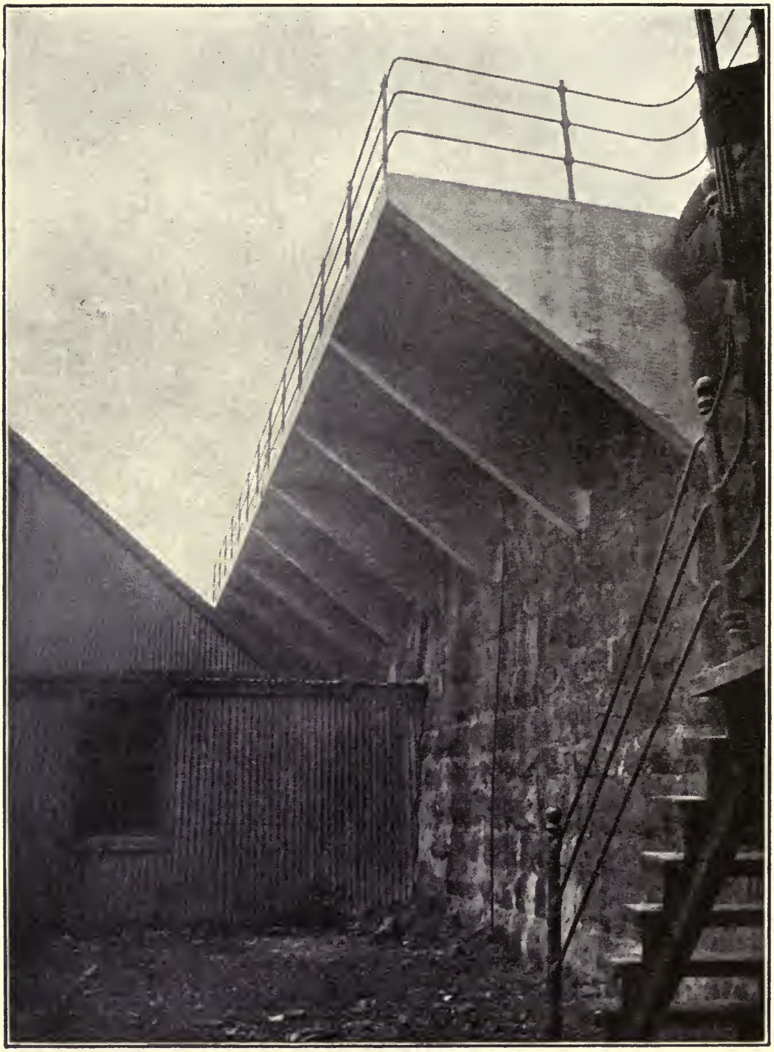
The swimming pond is 101 ft. long by 30 ft. wide, it is 7 ft. at the deep end, and 4 ft. at the shallow end, with gallery, steps, and a subway, all of concrete reinforced by expanded metal and steel rods. The pond bottom was made 1 ft. 6 in. thick on account of the bad ground met with. The walls are 9 in. thick with 1 ft. 9 in. buttresses placed at 10 ft. centres; the buttresses are continued to form beams to support the gallery steps. The treads of the latter are 4 in. thick, and the risers 3 in. reinforced by expanded metal. The inside of the pond is lined with 3 in. of fine concrete and Callender's bitumen sheeting, and finished with interlocking tiles.

Other Forms of Municipal Engineering Work.

Cantilever Platform.—The Bridlington Corporation have recently erected a reinforced concrete platform as illustrated in Figs. 174 and 175. This was designed by the borough engineer (one of the authors) and is being used as a fish stand. It is 50 ft. in length, and 9 ft. 6 in. in width, over all measurement. It is carried by reinforced concrete brackets which are 5 in. in thickness, of 4 to 1 concrete, and are spaced 6 ft. 2 in. centres. The platform diminishes from 8 to 4 in. in thickness, and is connected to a concrete wall which has been built at the back of the masonry wall of the harbour. The platform and wall are of 6 to 1 concrete.

The reinforcement consists of $\frac{3}{4}$ in., $\frac{7}{16}$ in., and $\frac{1}{2}$ in. rods, together with special clips and stirrups, and the work has been executed by the Chain Concrete Syndicate of Leeds upon their system. The aggregate consisted of sand and fine sea-gravel or ballast capable of passing through a $\frac{3}{4}$ in. ring, and the contract price was £114 12s. 6d. (this was exclusive of excavation, and cost of iron railings). On the completion of the work in December, 1910, a test load of 2 cwt. per square foot was applied.

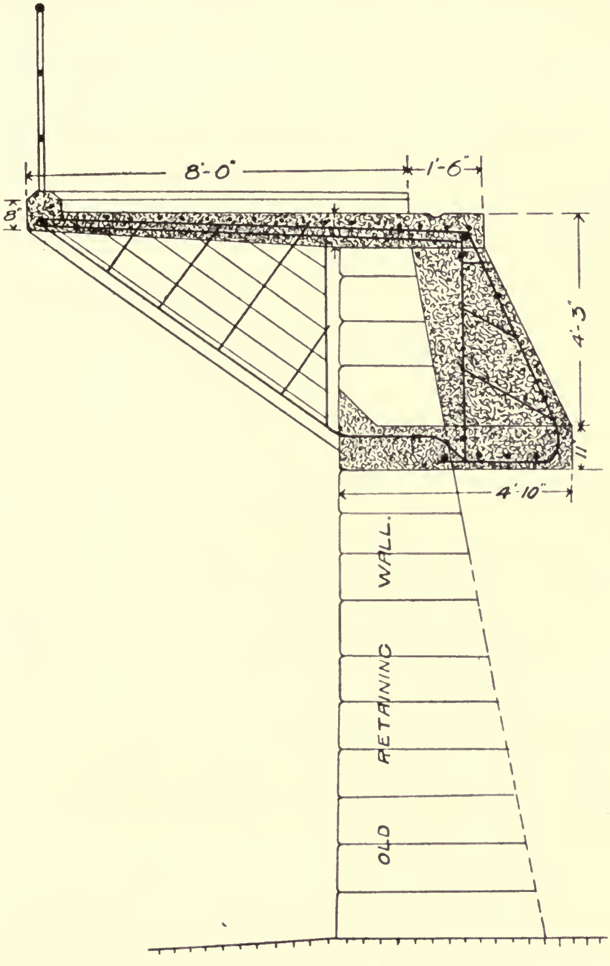
Public Shelters.—Reinforced concrete lends itself admirably to work of this description. One of the authors has designed for the Bridlington Corporation two reinforced shelters; these



"Concrete and Constructional Engineering."

FIG. 174.

are shown in Figs. 176 and 177, and are fully described in a paper read by him and which appeared in the "Transactions of the Society of Engineers" for August 1910. The shelters have each



"Concrete and Constructional Engineering."
FIG. 175.

an open front, reinforced concrete retaining walls at the back and ends, and reinforced concrete roof supported by reinforced concrete beams. Structures of this type can be erected 25 per cent cheaper than when the retaining walls are of brick or mass

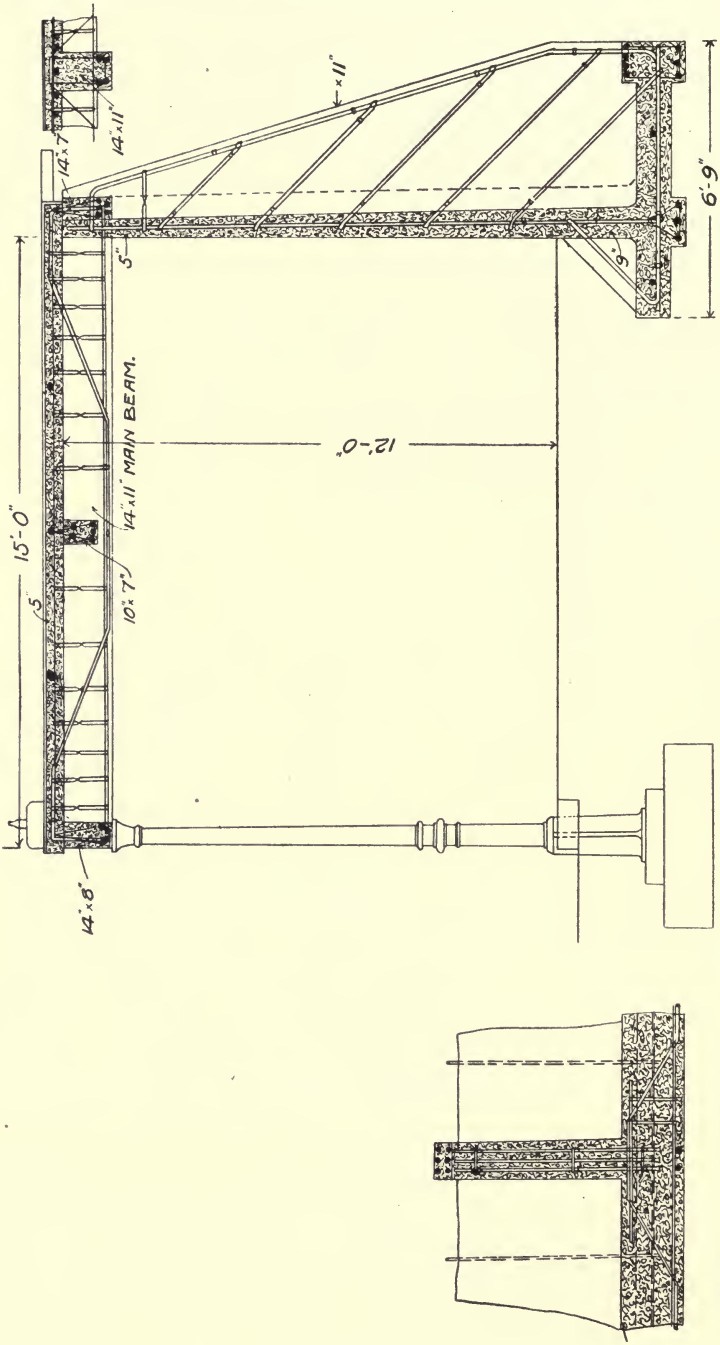


FIG. 176.

concrete, and the roof is of the old-fashioned steel joists and concrete type.

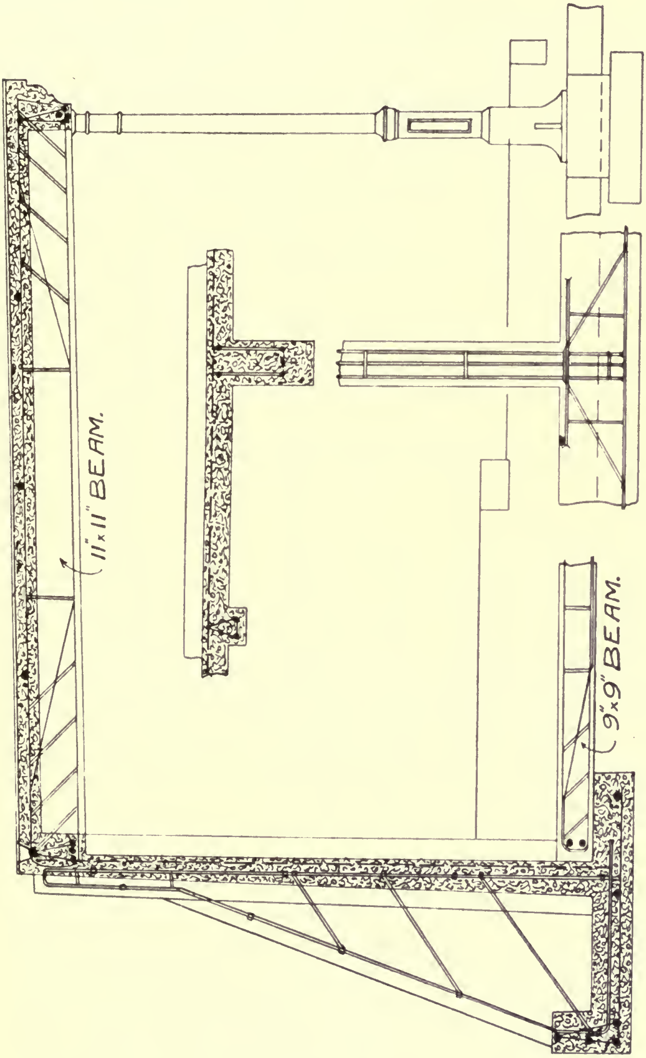


Fig. 177.

Underground Lavatories.—The material also lends itself to this class of work. One of the authors has erected an under-

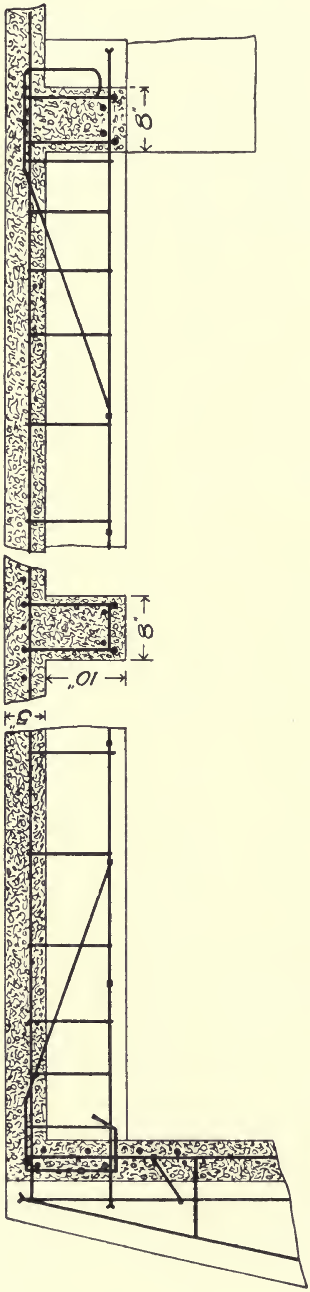


Fig. 178.

ground lavatory at Bridlington,[1] where reinforced concrete has been used in the construction of the retaining walls and roof (see Fig. 178) and a considerable saving has been effected thereby.

The work was carried out by the Chain Concrete Syndicate. He is now erecting two other public lavatories on the south side of the harbour.

The material has been used with great advantage for various other classes of municipal engineering work in this country, such as for highway bridges, retaining walls, boundary walls, sea defences, piles, dams, etc. Its use for some of these purposes is here illustrated.

The authors are of the opinion that reinforced concrete will be extensively used in municipal engineering work in the future seeing that the advantages of using this material are becoming so much better known.

[1] "Trans. Soc. of Engineers," August, 1910. Paper by E. R. Matthews, on "Reinforced Concrete Retaining Walls".

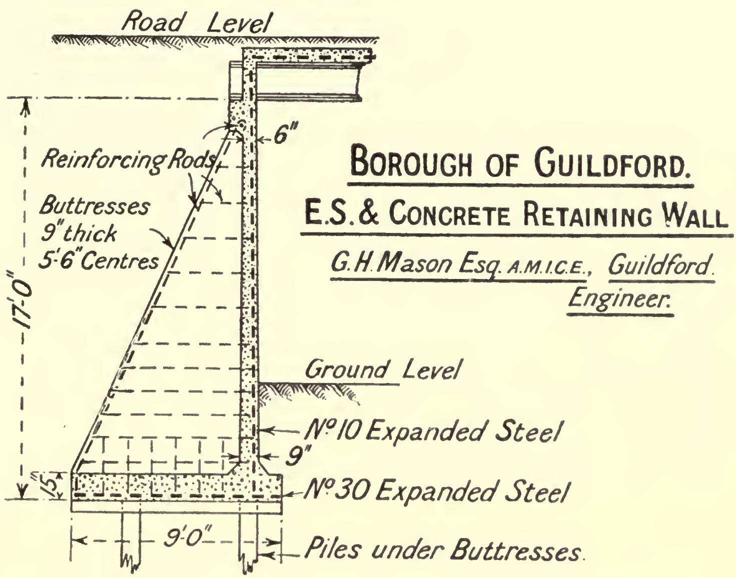


Fig. 179.

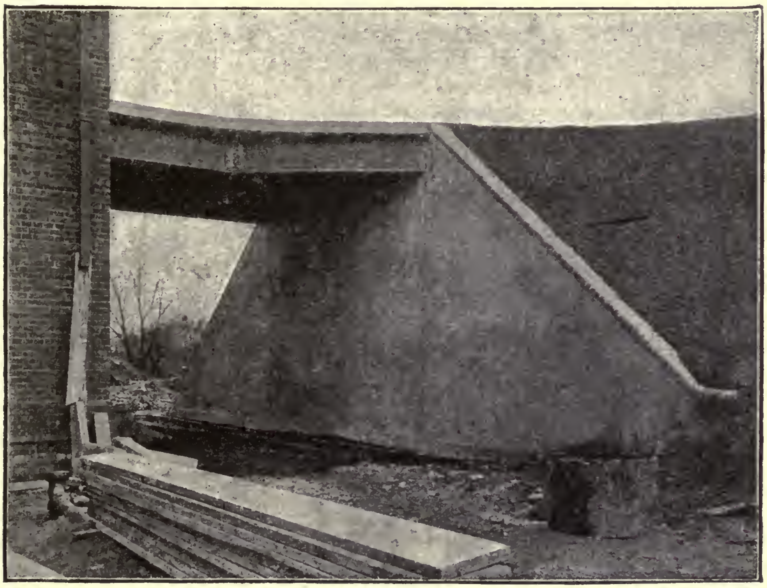


Fig. 180.

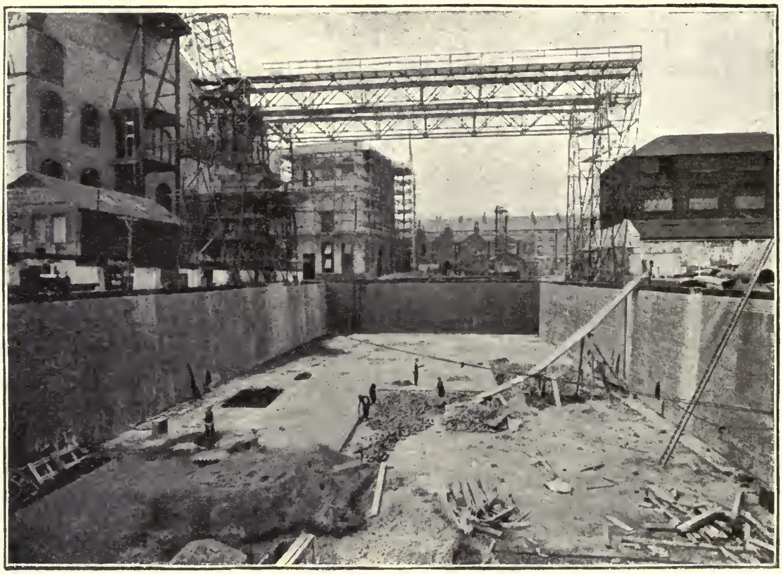


Fig. 181.

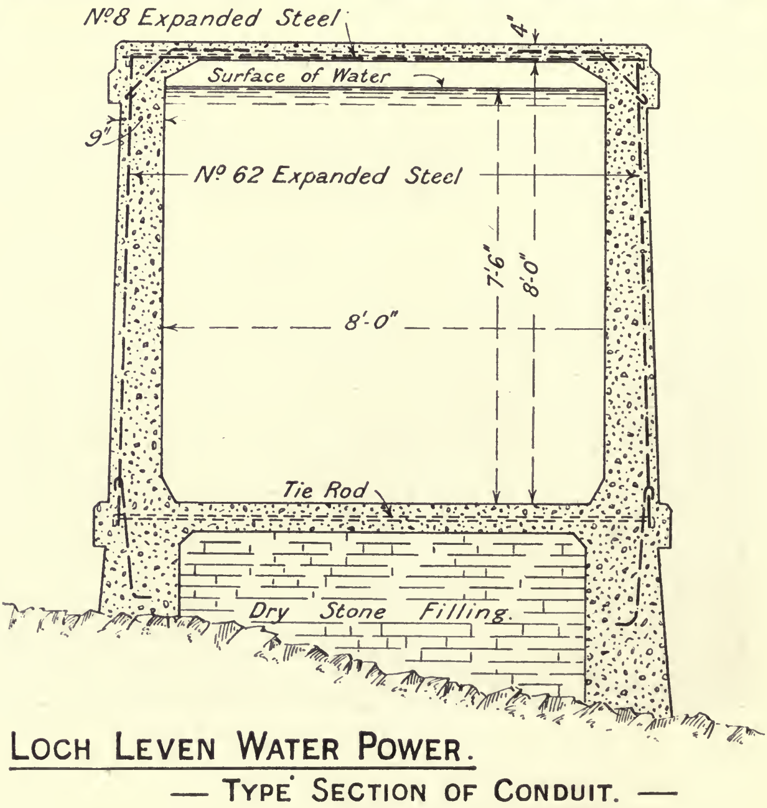


Fig. 182.

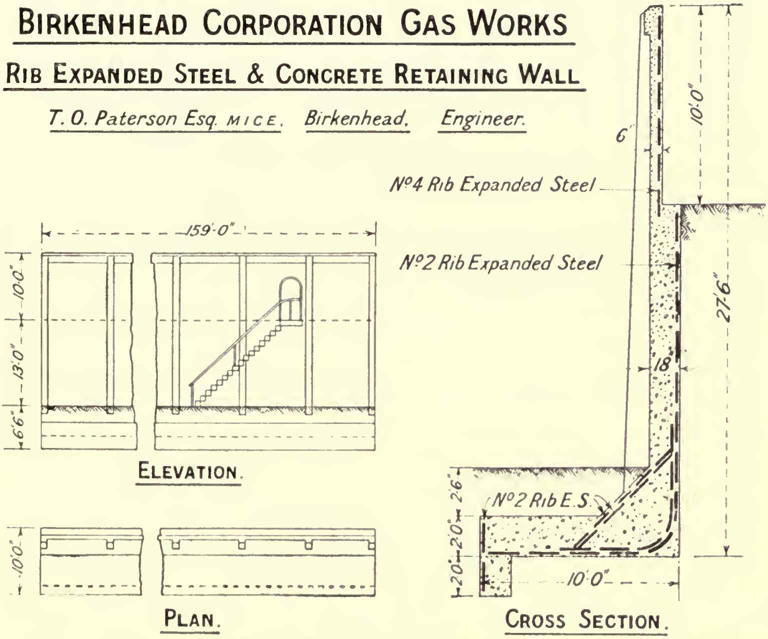


FIG. 183.

FIG. 184.

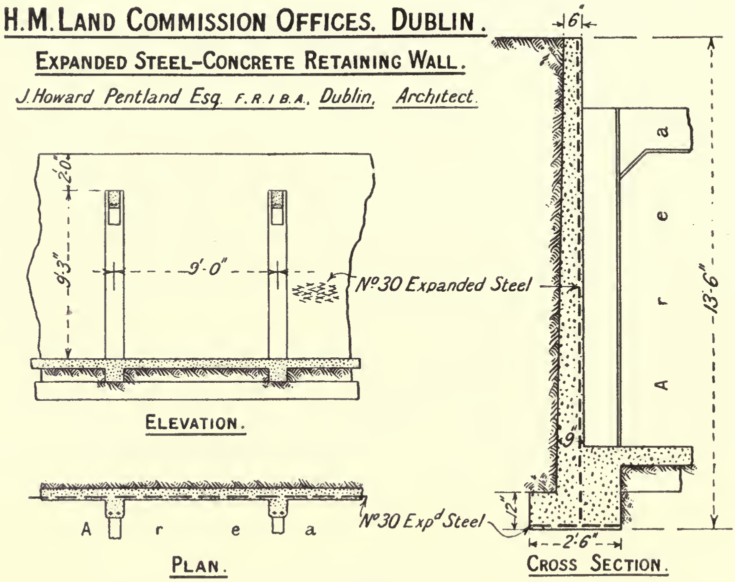


FIG. 185.

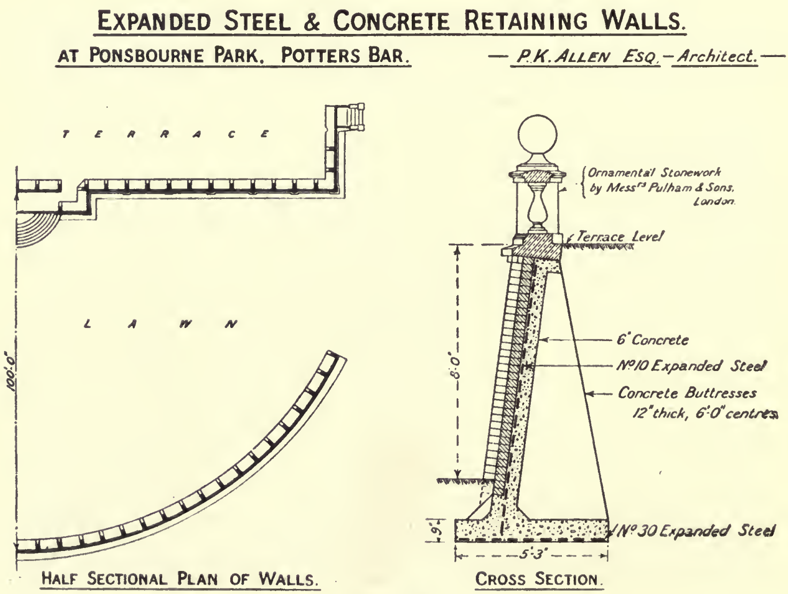


FIG. 186.

CHAPTER VIII.

REINFORCED CONCRETE IN RAILWAY ENGINEERING.

Reinforced concrete has been recognized by railway engineers in England, America, and on the Continent, as a material eminently suitable for use in the construction of bridges, retaining walls, bridge floors, box culverts, and other railway structures. Its use for purposes of this kind has been most extensive in America, and the authors propose first to give a brief description of some of the more important of the reinforced concrete structures on the Vandalia Railroad, U.S.A., the Wabash Railroad, U.S.A., and on one or two other American railroads, these representing typical examples of structures of this kind; they then describe the use of this material on English and Continental railways.

American Examples.

Reinforced Concrete on the Vandalia Railroad, U.S.A.—In an article on this subject which appeared in "Engineering,"[1] in 1910, the author describes some interesting bridges and culverts on this railroad. The following information and illustration are taken from that article:—

As prices of materials in England and in America differ so considerably, he gives the following as a unit for prices of materials, which will help in comparing the cost. The crushed stone used cost about 65 cents per cubic yard, the gravel 25 cents per cubic yard, the Portland cement 1·80 dollars per barrel, and the corrugated bars 2·10 dollars per cwt.

One of the principal bridges described was *The Eagle Creek Arch Bridge*. This is shown in Fig. 187.

[1] "Reinforced Concrete on the Vandalia Railroad," by E. R. Matthews, in "Engineering," 14 January, 1910.

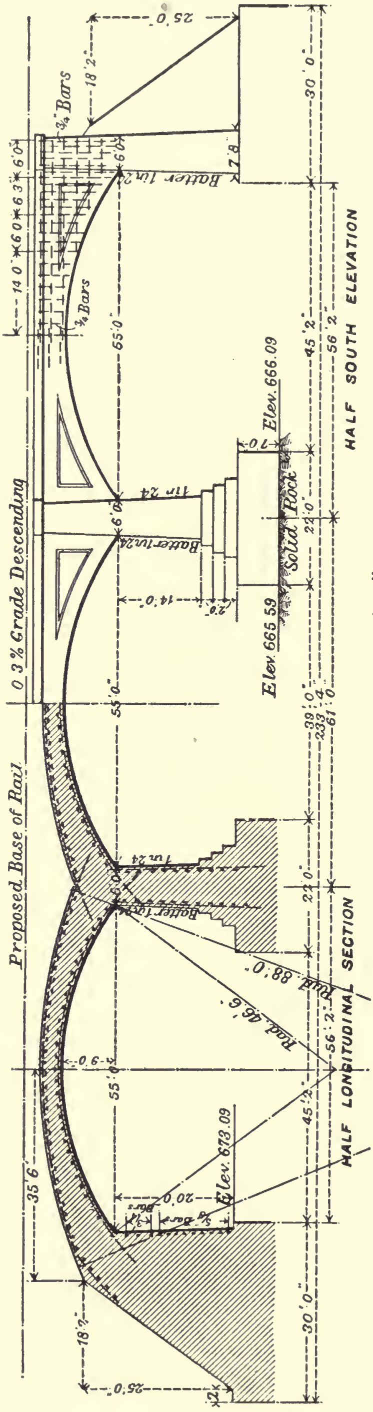


Fig. 187. "Engineering."

The bridge has three spans, each 55 ft., and carries three tracks. The arches were given but little rise, in order that the flood water should meet with as little obstruction as possible, and the piers were made slender in design for the same reason. The arches are designed to carry on each track a train load of 5000 lb. per lineal foot plus 60,000 lb. concentrated on a single axle, both uniform and concentrated loads being so placed as to produce maximum stresses when combined with dead load and temperature stresses.

The bridge is of concrete and reinforced concrete throughout, the mixture for the abutments and piers being 1 : 3 : 6, and in the arch ring and parapet walls 1 : 2 : 5.

Johnson corrugated reinforcing bars were used throughout.

Double-box Culvert at Station 3343.—This culvert has been built east of Seelyville, and is illustrated in Fig. 188.

A great many culverts of this type have been built by the Vandalia and other railroads in the U.S.A. The span of each opening is 20 ft. For any greater span than 20 ft. the archtype is preferable.

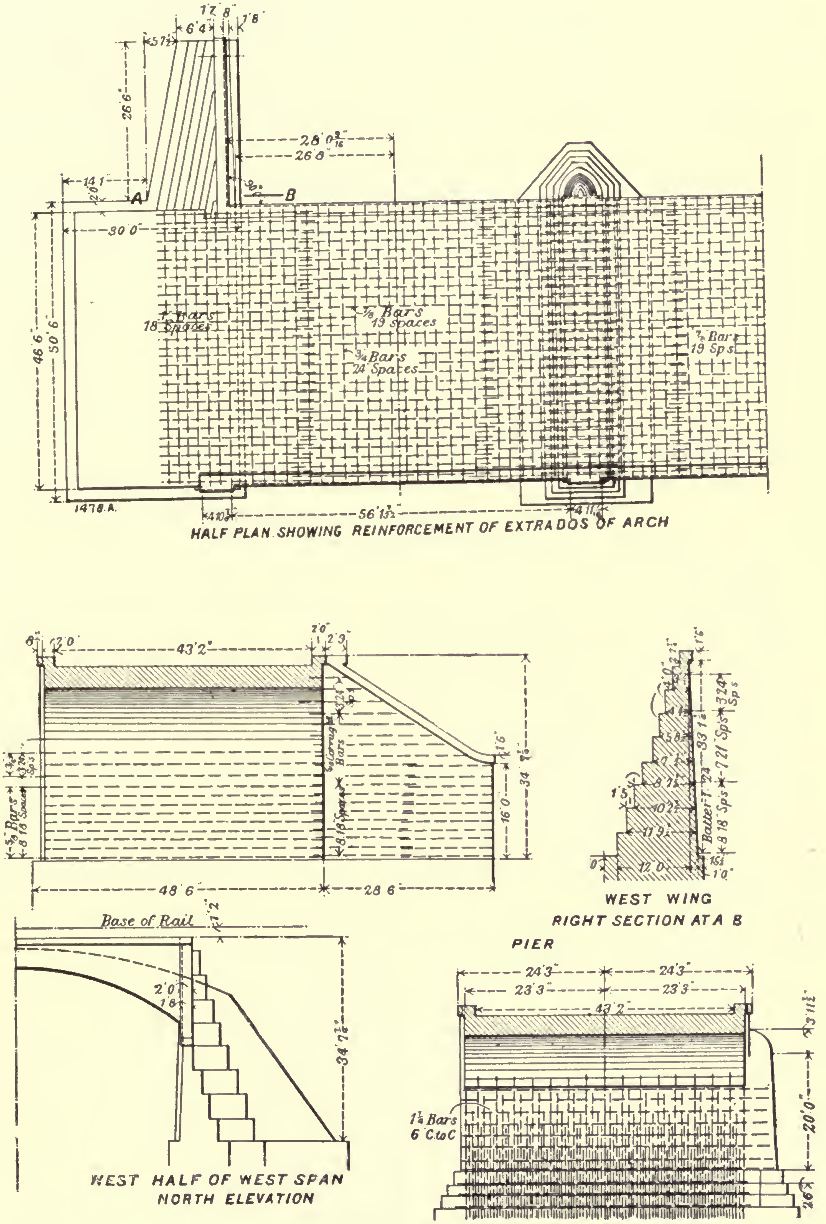


FIG. 187A. "Engineering."

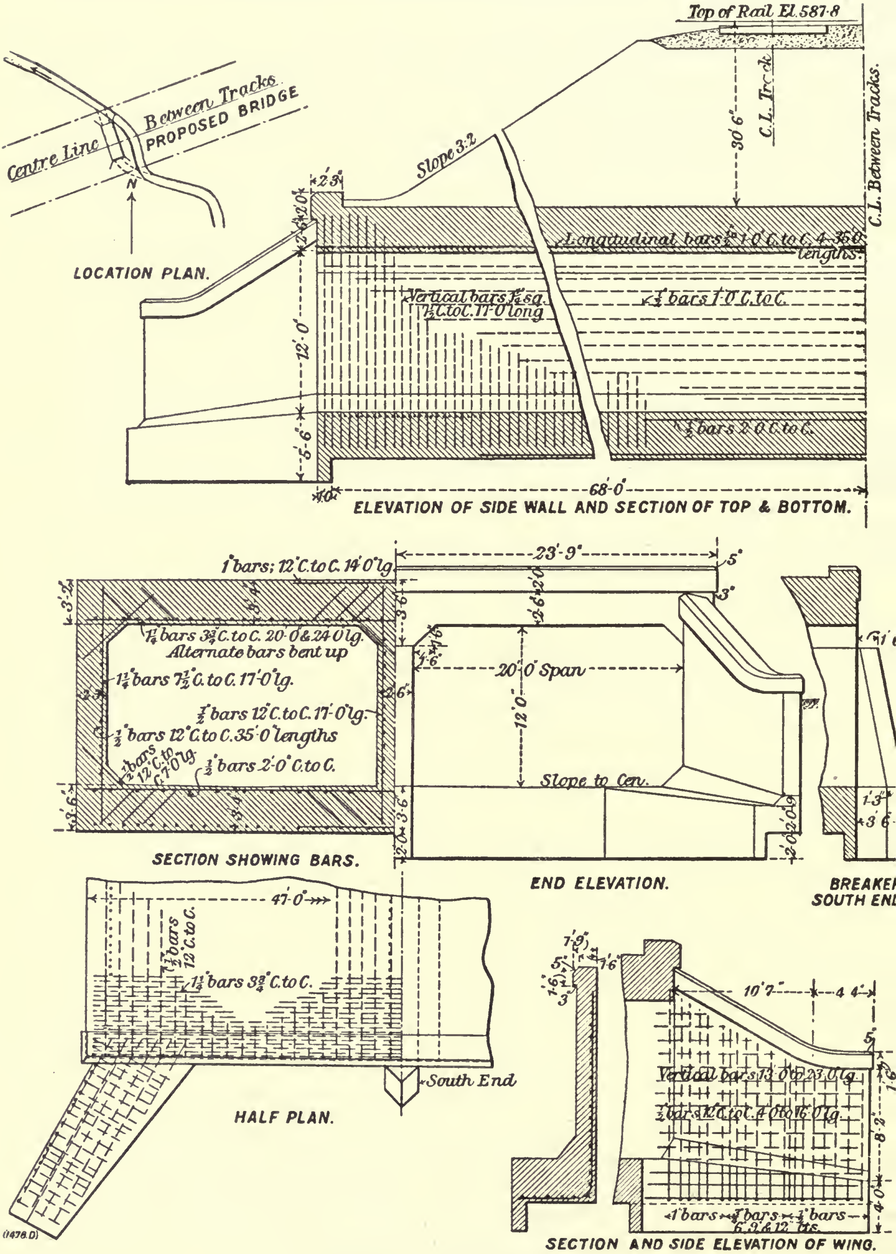


FIG. 188. "Engineering."

There is a 30 ft. embankment on the top of this culvert. The concrete mixture used was 1 : 2 : 5, the reinforcement consisted of Johnson corrugated bars.

Bridge at Seelyville. — This is shown in Figs. 189 and 190. It is built throughout of concrete and reinforced concrete, and has hollow reinforced abutments.

It carries a highway over four tracks of the Vandalia Railroad ; 1 : 2 : 5 concrete was used. The expansion joints are shown in Fig. 189. These interesting structures were designed by the engineer to the Vandalia Railroad Company, Mr. F. T. Hatch, M.Am.Soc.C.E.

Reinforced Concrete on the Wabash Railroad, U.S.A. —In a paper on this subject read before the Institution of Civil Engineers in April, 1910,[1] the authors describe a number of reinforced concrete structures which have been erected on the Wabash Railroad ; the following particulars and illustrations are taken from that paper, the Institution of Civil Engineers having kindly given permission for these to be used.

[1] "The Use of Reinforced Concrete on the Wabash Railroad, U.S.A.," by E. R. Matthews and A. O. Cunningham. "Proc. Inst. C.E.," Session 1909-1910.

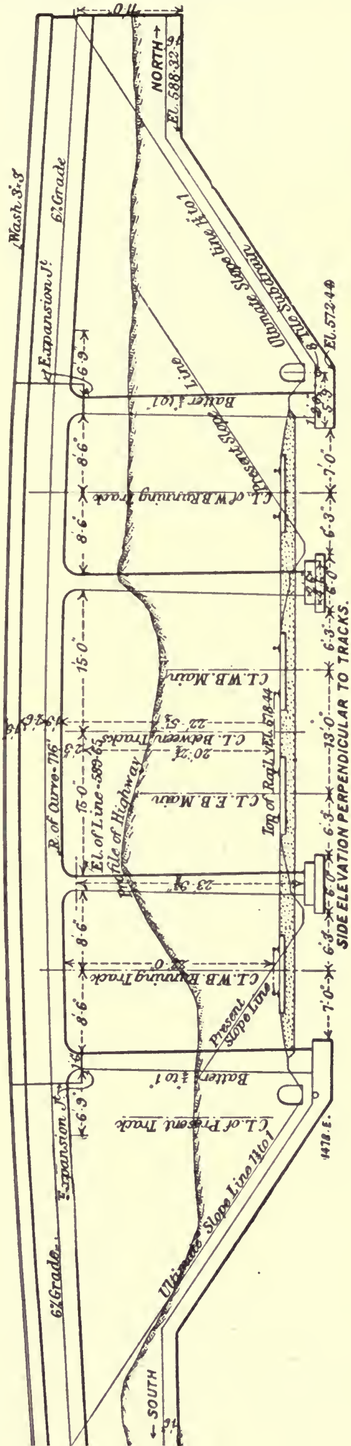


Fig. 189. "Engineering."

The Sangamon River Bridge.—(Figs. 191, 192, 193). "This is a double-track, four-arched skew-bridge over the Sangamon

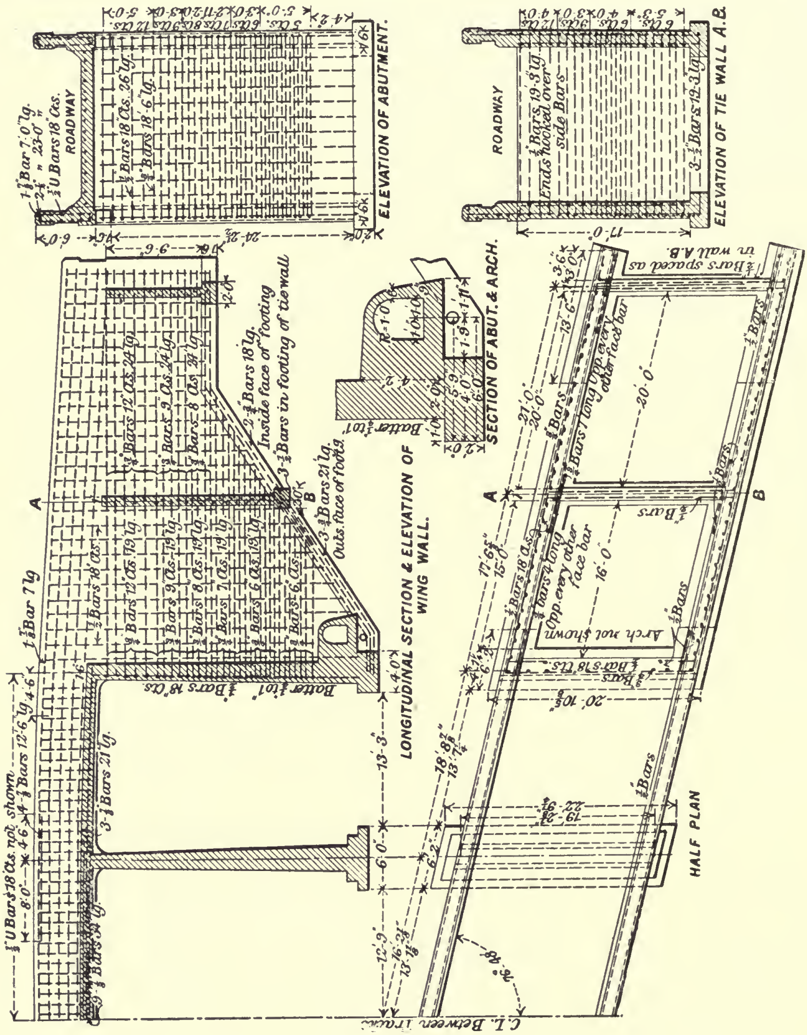


Fig. 190. "Engineering."

River. The abutments and arches are of reinforced concrete and the piers are of solid concrete, being reinforced at the base with

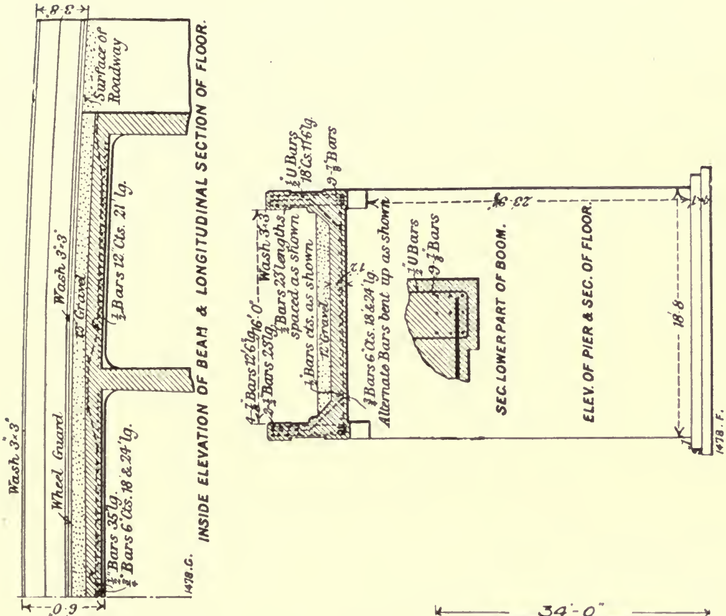


FIG. 190A. "Engineering."

1 in. square rods set $4\frac{1}{2}$ in. apart between centres[1] transversely. The toes are also reinforced with five 1 in. square rods placed longitudinally. The piers are carried on pile foundations, 259 round timber piles carrying each pier; these are driven into very stiff clay, and are spaced 3 ft. apart between centres longitudinally, and 2 ft. 6 in. transversely. The angle of skew is 45°;

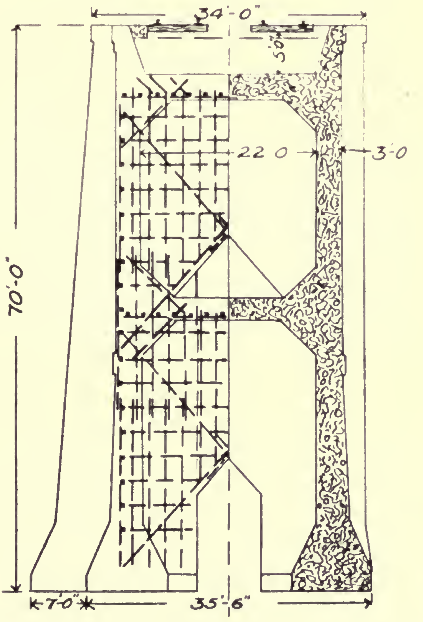


FIG. 191.

[1] Throughout this paper the distance apart of reinforcing bars denotes the distance between centres.

each arch has a semicircular square span of 61 ft. in the clear and a skew span of 100 ft. between centres of piers.

The arch-rings, which are 3 ft. 9 in. in thickness at the crown, are heavily reinforced with upper and lower ribs of 1 in. square rods 12 in. apart, crossed by 1 in. square rods about 4 ft.

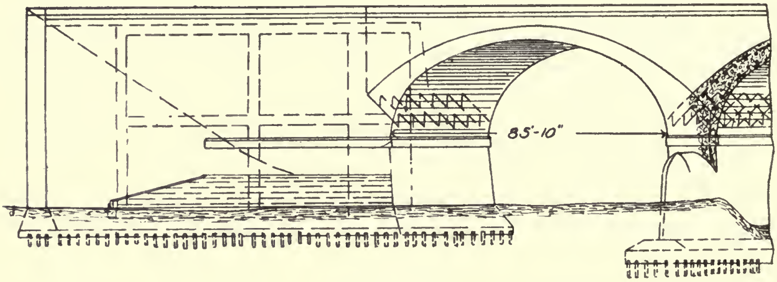


Fig. 192.

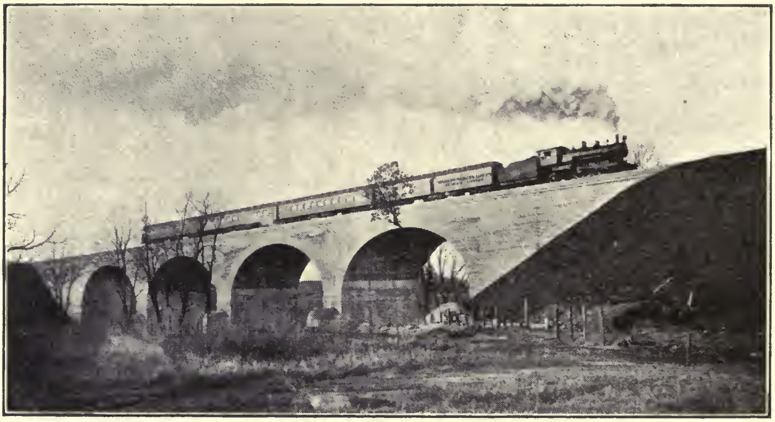


Fig. 193.

apart, laid parallel with the piers. The joints in the ribs are formed by overlapping the rods. The minimum cover over the reinforcement is 2 in., and the thickness of the concrete below the ribs in the arches varies from 6 in. at the crown to 24 in. at the springing.

"The features of particular interest in this bridge are the abutments and the method of providing square bearings for the arch ribs at the skew-backs. The abutments are practically boxes divided by horizontal and vertical partitions into four compartments. The side-walls, the partitions and the slab for carrying the road-bed are all 3 ft. thick. The side-walls are reinforced near their outer faces with vertical rods 24 in. and horizontal rods 36 in. apart. The slab for carrying the road-bed is reinforced with longitudinal rods 24 in., and transverse rods 4 in. apart, the ends of alternate rods being bent up at an angle of 45° for a length of 3 ft. 6 in. The lower slab is reinforced with longitudinal rods 24 in. apart, and transverse rods, all straight, 12 in. apart. The end-walls are reinforced with vertical rods 24 in. and horizontal rods 36 in. apart, and with six sets of upper and lower crossed bracing-rods spaced 6 in. apart in the thickness of the wall. Fillets reinforced with bars 12 in. apart are formed at the corners between the slabs and the walls. The parapet and spandrel-walls are reinforced with horizontal bars 24 in. apart placed near the face. The wing-walls for the bridge are reinforced with horizontal bars 24 in. apart. These wing-walls are provided in order to prevent scouring of the toe of the bank during high water, and to keep the bank from sliding. The height from the base of the rail to the bottom of the pier-foundation is 80 ft., and the height of the earthen filling on each end of the bridge is 60 ft. The cost of this bridge was $123,200 (£25,100)."

Forest Park Bridge, St. Louis.—(Figs. 194, 195.) "This bridge consists of concrete abutments carrying an 80 ft. double-track, through-girder bridge; the abutments are of the hollow type, namely, thin reinforced face-walls supported by counterforts resting on slab foundations. The principal features are the curved wing-walls, the concrete facia and balustrade carried by the outside girders, and the other ornamental concrete work. It will be noticed that the counterforts not only carry the girders, but also act as ties to hold the face-wall of the abutments. This face-wall is 2 ft. 8 in. thick at the bottom, and is battered in front so that the top thickness is 18 in. It is 18 ft. high and is surmounted by a slab 6 ft. wide and 18 in. thick, which serves as a bridge seat, and rests on top of the counterforts. On the back of this slab a cross-wall is built. The con-

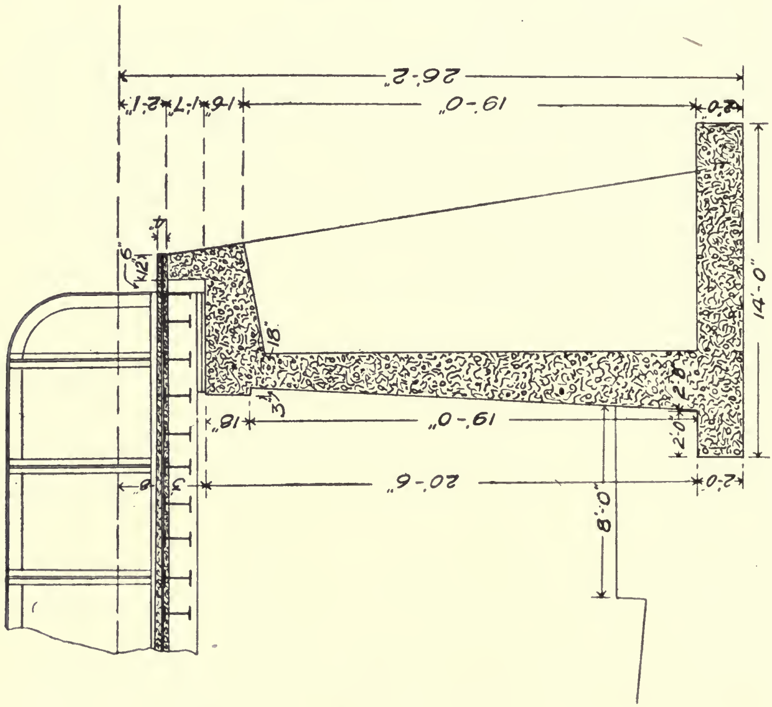


FIG. 194.

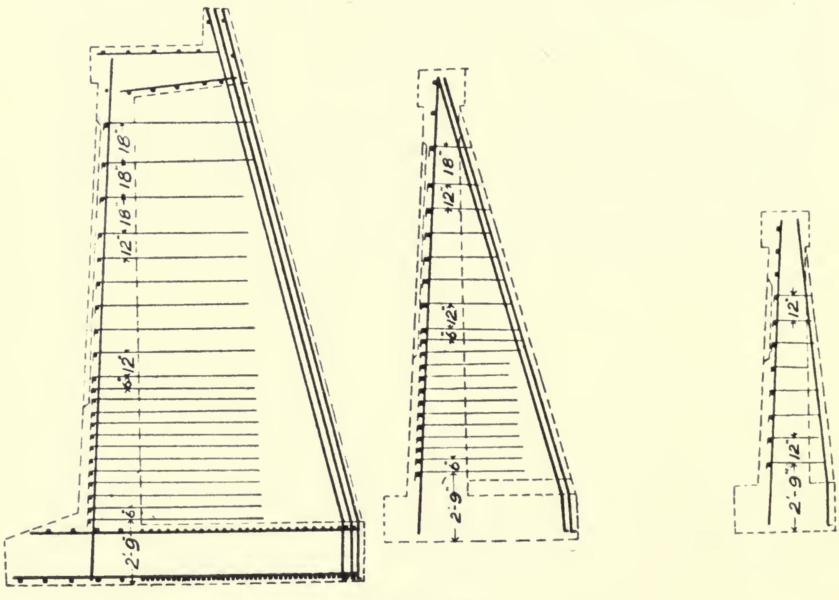


FIG. 194.

struction of the wing-walls is somewhat similar, though the top slab is omitted as shown. The widths of the counterforts are 3 ft. for the abutments and 18 in. for the wings, and the wall is thickened at its junction with the wings.

"The reinforcement in the abutments consists of ¾ in. square corrugated rods placed at the following distances apart:—

"For the face-walls, vertical rods, 24 in.; horizontal rods, 6, 12, or 18 in. at different levels.

FIG. 195.

"In the bridge-seat, the longitudinal rods are 12 in., and the transverse 24 in. apart; and for the counterforts, the horizontal rods, 6 ft. long, are 12 in. apart, and the vertical rods 6, 12, or 18 in. at different levels.

"The tie-rods in the back of the middle counterfort are eighteen in number, placed 6 in. apart. The end counterforts have nine tie-rods. The reinforcement in the wing-walls is similar to that in the abutments except that the rods in the foundation slabs are all spaced 12 in. apart.

"The bridge floor is of concrete 8 in. thick, and is reinforced in both faces with ½ in. corrugated bars. This slab is carried upon 15 in. steel I beams, which are riveted by connecting angles to the webs of the girders, and are spaced 18 in. apart between centres. In each abutment 28,700 lb. of ¾ in. corrugated rods were used.

"Reinforced concrete is sometimes objected to on the ground that ornamental structures cannot be built of this material; but the authors venture to claim that this bridge contradicts such a statement. It is built at the main entrance to the largest and most beautiful park in St. Louis: hence the necessity for æsthetic treatment.

"This substructure and the ornamental work was constructed by contract, but the railway company erected the steel work themselves; the cost was $22,307 (£4565), but this included the removal of an old bridge and its stone substructure; also a large amount of excavation on account of increasing the width of the opening, and the necessary temporary bridging to provide for the traffic. As this bridge was built during the St. Louis World's Fair, and as more than 300 trains crossed it per day, it was impossible to remove the extra earth and the old bridge with trains, so that it was necessary to do this with teams at a largely added cost."

Standard Reinforced Concrete Box Culverts.—"These cul-

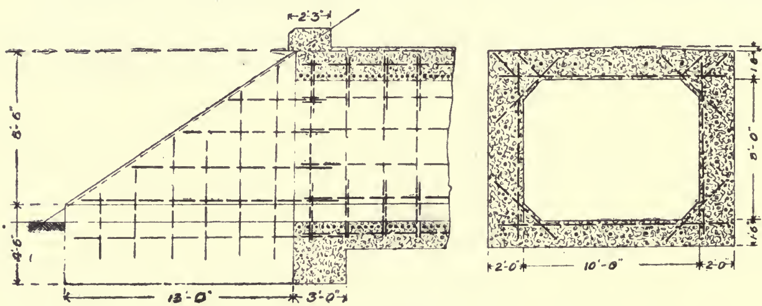


Fig. 196.

verts are used for comparatively small openings, being designed for spans of 4, 5, 6, 8, 10, 15, and 20 ft. respectively.

"One of the 10 ft. culverts is shown in Fig. 196. The bottom slab is 1 ft. 6 in. thick, the top slab is 1 ft. 8 in., with a slight gradient from the centre each way, and the walls are 2 ft. thick. The reinforcement in the bottom slab consists of horizontal longitudinal bars 24 in. apart, and transverse bars (i.e. parallel to the tracks) 6 in. apart; diagonal bars are inserted at the junction with the walls. The walls are reinforced by horizontal and vertical bars 24 in. apart, and the top slab in a similar manner to the bottom one, except that the transverse bars are placed on the under and not the upper side, and that the ends of alternate bars are bent up at an angle of 45 degrees for a length of 1 ft. 9 in. All reinforcement has at least a 2 in. covering of concrete, and the bars used are $\frac{7}{8}$ in. square; the cement had to pass the standard test adopted by the American Railway Engineering and Maintenance of Way Association in its 'Manual of Recommended Practice' of 1905. In a box culvert of this size 2·944 cub. yds. of reinforced concrete is inserted per lineal foot of barrel, and 50·8 cub. yds. in the two caps, two aprons, and four wings."

Size in Feet.	Cost per lineal foot of barrel.		Cost of Wings, Caps and Aprons.	
	dollars.	£ s. d.	dollars.	£ s. d.
4 × 3	7·50	1 10 6	60·00	12 5 0
5 × 4	9·50	1 18 9	104·50	21 7 6
6 × 5	12·00	2 10 0	132·50	27 0 0
8 × 7	20·50	4 3 6	257·00	52 10 0
10 × 8	25·50	5 4 0	380·00	77 15 0
15 × 10	48·50	9 18 0	562·00	114 10 0
20 × 10	77·00	15 14 6	657·00	134 0 0

Culvert No. 24.—" This is a 20 ft. by 10 ft. box culvert of the foregoing type; it is built for a double track. The wing-walls are built slightly on the splay, and slope down at an angle of 45° in elevation. A view of one is given in the figure. They are reinforced by $\frac{3}{4}$ in. horizontal bars 24 in. apart, and $1\frac{1}{4}$ in. vertical bars 12 in. apart. The width of these walls at the base varies from 2 ft. 6 in. to 5 ft. 6 in., and the base is reinforced with $\frac{3}{4}$ in. rods 6 in. apart; the walls are battered at front and back 1 in. to 1 ft. and are surmounted by a reinforced concrete coping."

All concrete used in these structures was mixed by machinery. Expansion joints were left where required. The works were designed by Mr. A. O. Cunningham, M.Am.Soc.C.E., the chief engineer to the Wabash Railroad Company, and carried out under his supervision.

The prices of materials, etc., were as follows:—

Gravel, 50 cents per cubic yard at pit.

Fig. 197.

Cement (varies) average 1 dollar per barrel at the cement plant.

Sand, 35 cents per cubic yard at pit.

Reinforced bars, about 2 dollars per 2000 lb.

Wet excavation, 1 dollar per cubic yard.

Dry excavation, 50 cents per cubic yard.

Plain concrete, 5·25 dollars per cubic yard.

Reinforced concrete, 7 dollars per cubic yard.

Other American Examples.

Vermillion River Bridge.—This has been erected for the Cleveland, Cincinnati, Chicago, and St. Louis Railway Co. It was designed by the engineer to that Company, and built by Bates and Rogers Construction Co., of Chicago, in 1905. It is a fine and graceful structure, constructed throughout of concrete and reinforced concrete. It is illustrated in Fig. 199, while Fig. 198 represents the false work, or centering, for main arches, and

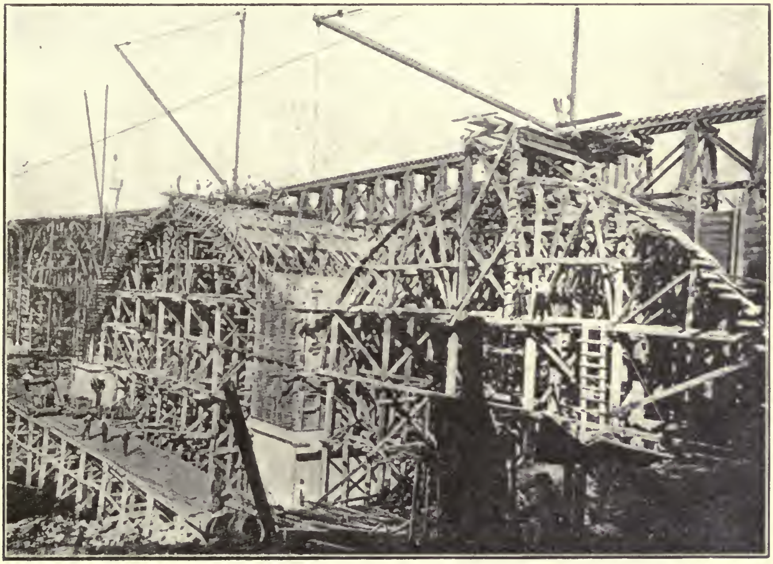


Fig. 198.

the location of the derricks. The bridge consists of three arches, the central span being 100 ft., and the two side spans 80 ft., with rises of 40 and 30 ft. respectively.

The arch rings are 3½ ft. thick at the crown, deepening out towards the springing lines, and are reinforced near the extrados and intrados with 1 in. corrugated bars 12 in. apart and overlapped 4 ft. at their ends. Between these rods at the extrados, and above them at the intrados, there is a series of $\frac{7}{8}$ in. transverse bars 33 ft. long.

The channel piers are hollow, the pilasters being carried up as reinforced facing slabs 15 ft. wide and $3\frac{1}{2}$ ft. thick. The transverse walls are formed by the piers of the spandrel arches next to the springings, which have brackets at the top projecting

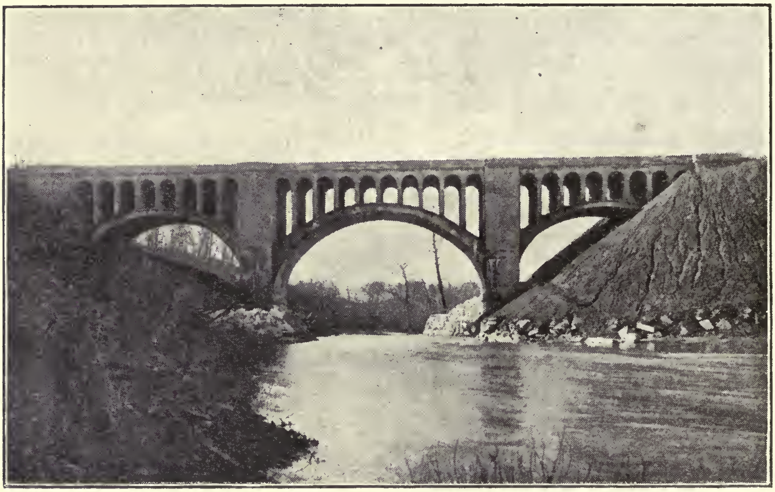


Fig. 199.

12 in. on the inside; these brackets carry reinforced concrete slabs 2 ft. thick, which, being freely supported on rails embedded in the top of the piers, and bearing against similar rails projecting from the underside of the slabs, act as expansion joints. A similar transverse expansion joint is placed over the top of each abutment.

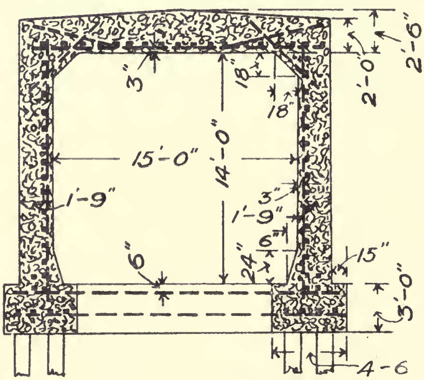


Fig. 200.

The concrete for the reinforced portions was mixed in the proportions of 1 : 2 : 4 (cement, sand, broken stone); that for the abutments and main piers of 1 : 3 : 6, and the footings of 1 : 4 : 8.

Indian Creek Culvert.—This is shown in Figs. 200, 201, 202,

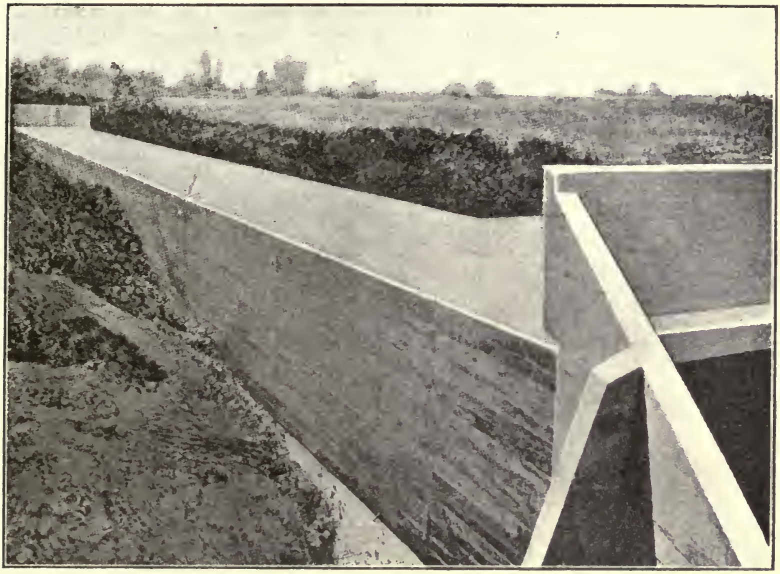


FIG. 201.

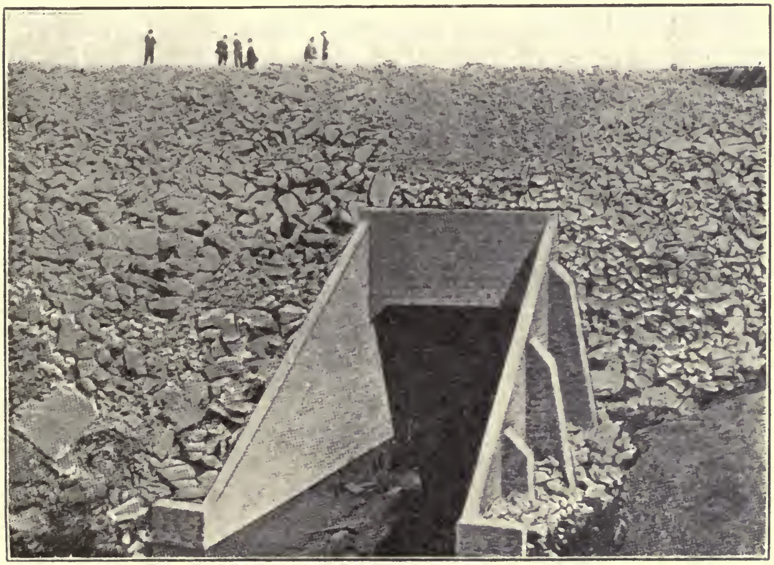


FIG. 202.

which represent the completed structure before and after filling. The culvert has been constructed on the Kansas City, Mexico, & Orient Railway. It is 14 ft. by 15 ft. by 250 ft. long.

The concrete used was in the proportions of one part cement to three parts Kansas River sand, and five parts crushed limestone; the mixing was done by a No. 1 rotary mixer, and corrugated bars were used as the reinforcement.

The structure was designed by Mr. W. W. Colpitts, M.Am.Soc.C.E., assistant chief engineer of the Kansas City, Mexico and Orient Railway, and was built by Mr. L. J. Smith, general contractor, of Kansas City, in 1905.

Reinforced Concrete Roundhouses in America.—Many of these have been erected. One of the best examples may be found on the Santa Fé Railway. This structure is one of the largest of its kind in the United States, and was designed by Mr. Harrison Albright, the supervising architect. It is 850 ft. by 94 ft. with 35 stalls, each being 92 ft. long. The inner circle is 14 ft. wide, the outer circle 25 ft. 6 in. The columns vary in size from 18 in. by 40 in. to 22 in. by 39 in. The roof slab is 4 in. thick, and is supported by longitudinal beams and roof girders. The outer walls are 9 in. thick.

An interesting description of this building appeared in "Concrete and Constructional Engineering" in October, 1910, from which the view is taken.

Stations and Platforms.—Reinforced concrete has also been used extensively for this purpose, and has given entire satisfaction.

An interesting example may be found in the *Marathon Station* which is shown in Fig. 204.

This material has also been largely used for signal towers, power houses, warehouses, retaining walls, grain bins, wharves, telegraph poles, foundations, strengthening old masonry, etc., but the authors deal with its use for most of these purposes in subsequent chapters. It has also been used in a few cases in tunnel construction, one of the earliest examples being the Aspen Tunnel, Union Pacific Railroad, constructed in 1901. A portion of this tunnel, 713 ft. in length, was constructed of reinforced concrete. The expensive work of maintaining a brick-lined tunnel is avoided when this material is used.

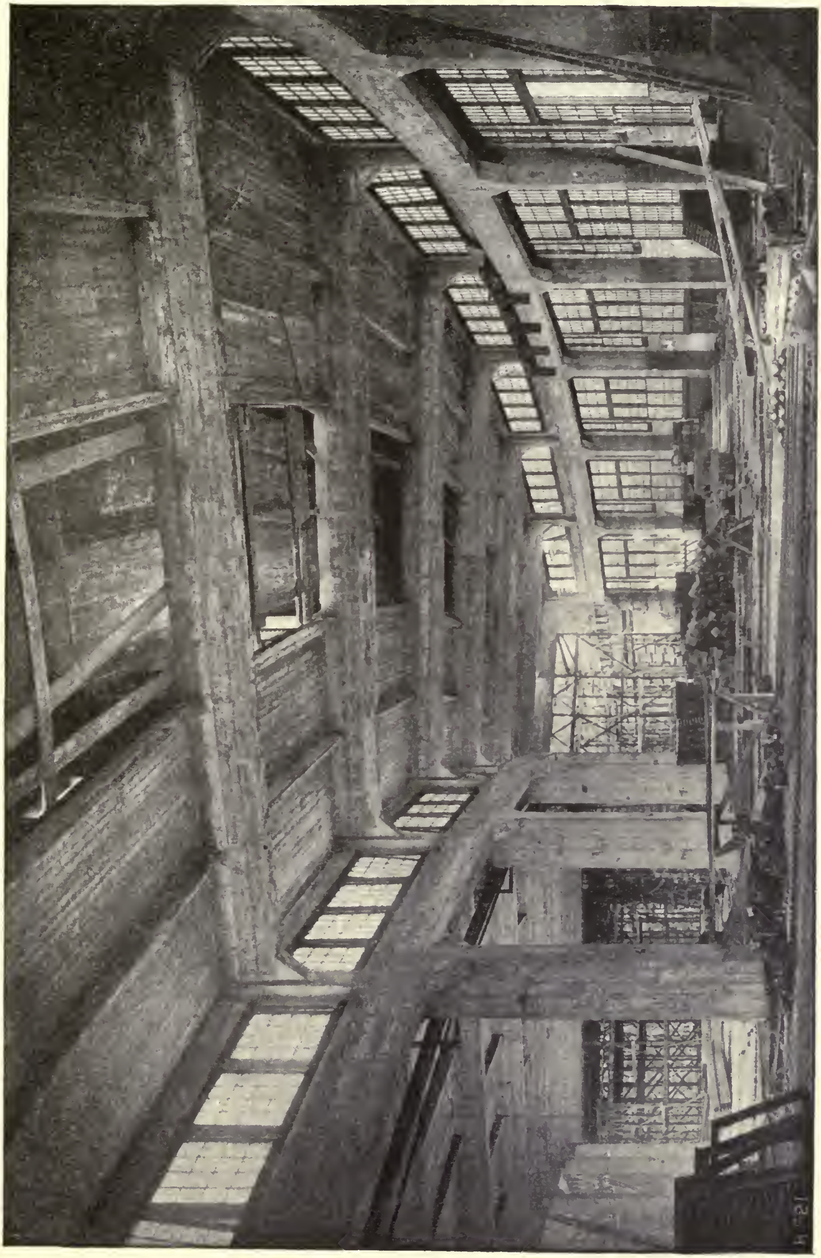
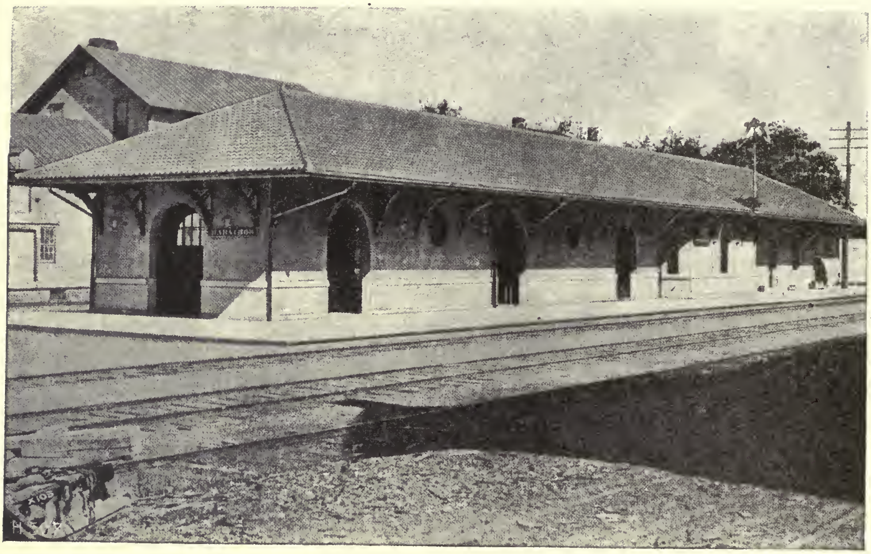


Fig. 203.

"Concrete and Constructional Engineering."



"*Concrete and Constructional Engineering.*"

FIG. 204.

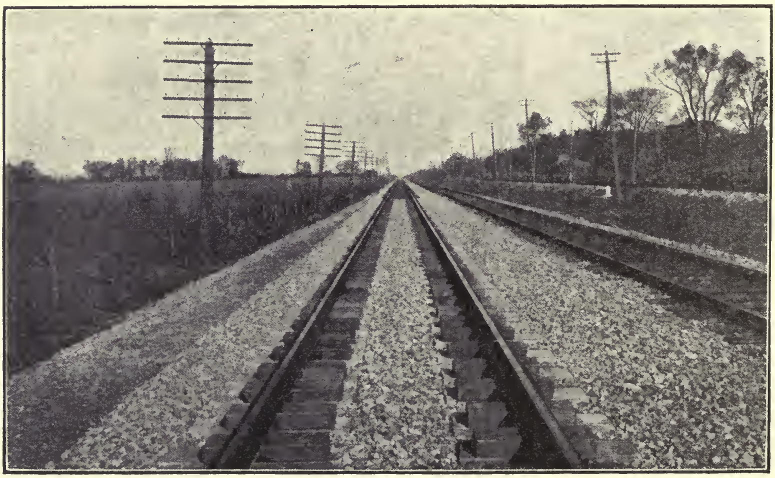


FIG. 205.

Railway Sleepers.—It has been estimated that in the United States approximately 118,000,000 sleepers are used annually by the railway companies.[1] During the past few years experiments have been made on many of the railways to ascertain the relative efficiency of reinforced concrete sleepers compared with those of timber. As these tests have not, in the authors' opinion, been carried out for a sufficiently long period to be conclusive, no opinion is given on the matter.

Ties of this type are now in use on the International Railway, Buffalo, the Chicago and Alton Railway (see view), and many other American railroads.

Concrete Roadbed.—The N.Y.C. and H.R.R. Railway, are now putting down as an experiment a solid concrete roadbed, which is undoubtedly excellent for subways where a minimum of dust is essential.

English and Continental Examples.

Bridge on G.E.R.—Figs. 206, 207, 208 represent a bridge constructed at Angel Road Station, Tottenham, for the Great Eastern Railway Co., it has seventeen spans of 42 ft. 9 in. each; the width between parapets being 40 ft. The view (Fig. 207) shows the expansion joints on the curved portion of the bridge. The structure was designed and carried out by the Considère Construction Co.

Bridge at Chateau Thierry.—This is an arch bridge (see Fig. 209), and a very graceful structure. In the design of structures of this class the Considère system is very successful. Use is made of the "influence lines" of Prof. Weyrauch in plotting the moments generated by the various positions of the moving load. In flat arches, temporary hinges of spiralled concrete are introduced at the crown and springings to ensure that there shall be no unknown initial stress in the arch due to some slight settling of the abutment, or the shrinkage of the concrete in setting. This method has been used by M. Considère many times on the Continent, and is now being introduced into the bridges on this system at Warrington and Bridgend.

The view of the bridge was taken before concrete was filled

[1] "Concrete and Constructional Engineering," May, 1910, p. 335.

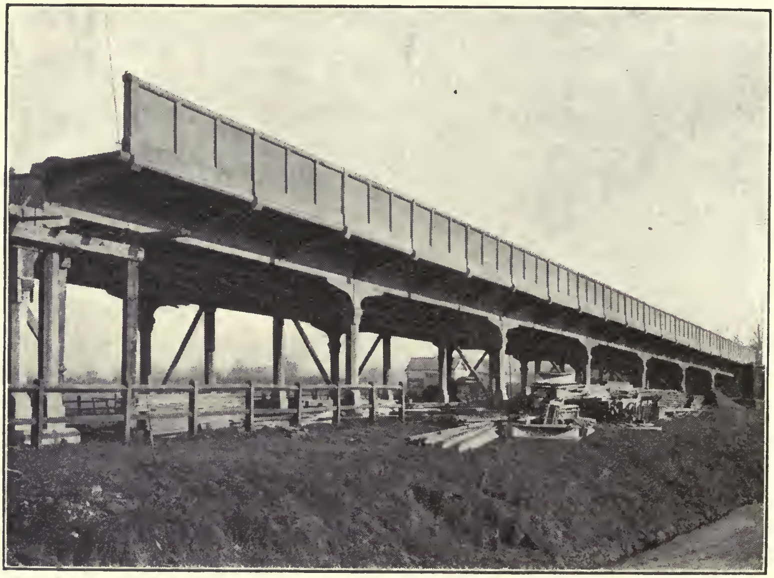


Fig. 206.

Fig. 207.

Fig. 208.

Fig. 209.

in the hinges at springing. The bridge has two spans of 86 ft. each.

Bridge at Immingham Dock.—Fig. 210 represents a bridge recently erected at Immingham Dock. It is constructed throughout of reinforced concrete, and was built on the Hennebique system.

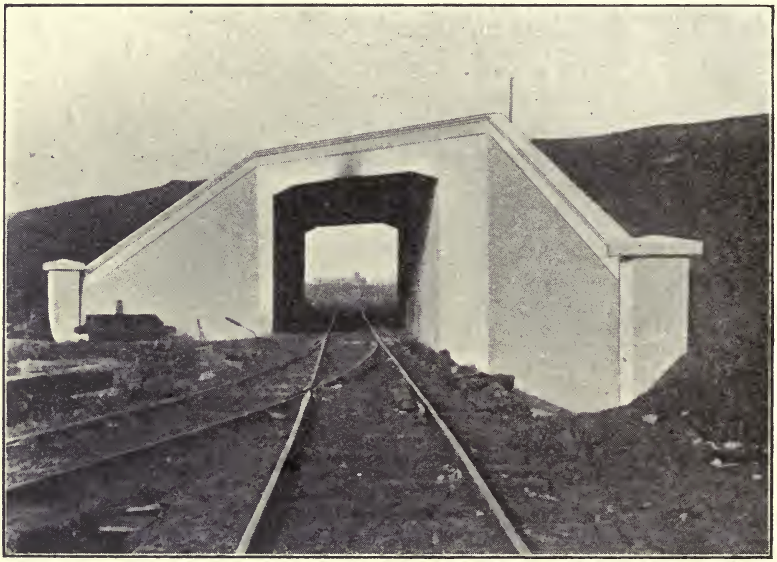


Fig. 210.

Luggage Subway.—Fig. 211 shows a luggage subway which has been constructed in reinforced concrete for the N.E.R. Co., under the main line at Middlesborough Station. It was carried out on the Hennebique system.

Railway Bridge over River See at Avranches, France.—Figs. 212, 213 represent this bridge. Its total length is 281 ft. and it is of the bowstring girder type, the span shown being 100 ft. It is constructed throughout on the Considère system, and is a useful structure.

Skew-bridge near Paris.—This is shown in Figs. 214, 215. It is a bridge that has to bear a considerable amount of heavy road traffic; it is built over a double track. The work was subjected to some severe tests by the Engineers of Bridges and Ways. The span measures 60 ft., and the width of bridge is about 36 ft. It is built on the Coignet system.

Railway Viaduct at Gennevilliers Gas Works.—This is represented in Fig. 216. The viaduct bears four railway tracks. A considerable saving was effected by using reinforced concrete

Fig. 211.

in this structure, and the work was carried out in only a few months. It is also constructed on the Coignet system.

Bridge at Luxemburg.—This bridge has a span of 250 ft. The deck platform, which bears a considerable amount of heavy traffic, including a steam tramway, is supported by reinforced beams. The bridge was designed by M. Sejourné, Chief Engineer of Bridges and Ways, and was built under his supervision. It is also on the Coignet system.

It will be seen from the foregoing illustrations, that some

Fig. 212.

Fig. 213.

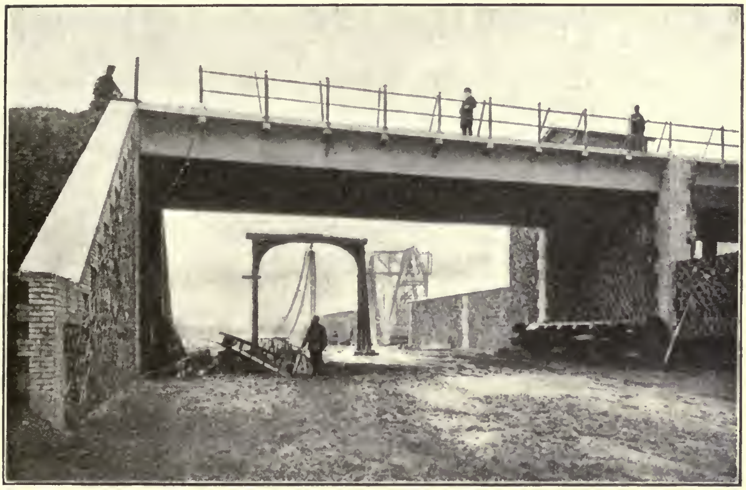


Fig. 214.

Fig. 215.

very important railway engineering work has been carried out in this material, and there is every likelihood of it being more extensively used in the future on British railways, for the dur-

Fig. 216.

ability of the material, freedom from vibration, fire resistance, and water-tightness render it a material unequalled by any other.

CHAPTER IX.

REINFORCED CONCRETE IN THE CONSTRUCTION OF WHARVES, JETTIES, GROYNES, SEA-WALLS, BINS, FACTORIES, AND OTHER ENGINEERING WORKS.

In no class of engineering work has this material been more successfully used than in the construction of harbour and dock works, such as quays, wharves, and jetties.

As some very important work of this class has been done in this country, the authors intend to say little regarding similar work which has been carried out on the Continent and in America.

Perhaps no firm has executed more work of this kind than the Hennebique Contracting Co., who really make a speciality of it. The first examples given will therefore represent some of the harbour works carried out on the Hennebique system.

Piling.—The authors intend to deal fully with this in the next chapter, namely, the chapter on Building Construction; they will therefore only incidentally refer to it here.

Extension of Quay, Prince's Dock, Liverpool.—(See Figs. 217-220.) This work includes the driving of 236 piles 16 in. × 16 in., and about 35 ft. long; also bracings, beams, and decking, all in reinforced concrete. The length of the extension is 713 ft., width 65 ft. The work has been carried out for the Mersey Docks and Harbour Board, the engineer being Mr. A. G. Lyster, M.Inst.C.E.

Fig. 217 is a view showing the piles, beams, and bracings;
Fig. 218 shows the construction of the decking; and
Fig. 219 the driving of the piles.

Waterford North Viaduct.—This viaduct on the G.S. and W. Railway, Ireland, is an interesting piece of engineering work, and the view here given (see Fig. 221) is an unusual one, for it

FIG. 217.

FIG. 218.

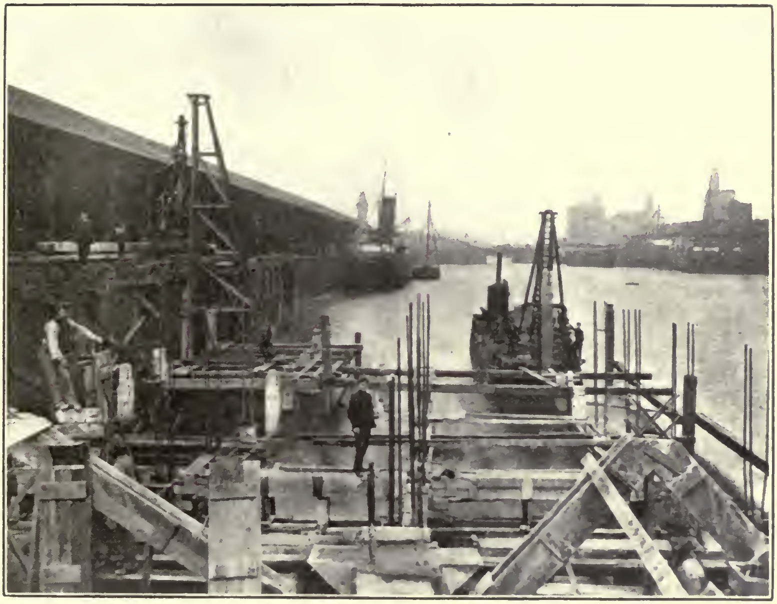


Fig. 219.

Fig. 220.

represents one of the longest reinforced concrete piles in the world being driven. The pile is 62 ft. long and 16 in. × 16 in.; it weighs 8 tons.

The length of the viaduct is 720 ft., width 35 ft., total number of piles driven 106.

Fig. 221 represents the driving of a 62 ft. pile;

Fig. 222 is a view from underside showing the piles, beams, and bracing; while

Fig. 223 shows the reinforced concrete parapet walls.

Underpinning of Pier, Newhaven.—Some good work of this class has also been carried out on the Coignet system. The authors have only space to give one example, and that refers to the underpinning of the West Pier, Newhaven.

This work was carried out for the London, Brighton and South Coast Railway Co., and was designed by their engineer, Mr. Chas. Morgan, M.Inst.C.E., the contractors being Messrs. W. Hill & Co., of London.

About 600 piles were used, and connected to the masonry of the pier.

It was necessary to replace the old wooden piles supporting the work, on account of the decayed condition of the wood. Guide piles were driven; these were reinforced with four principal bars of angular section, bound together by flat iron ties, the entire framework being bound with wire. The concreting operation was carried out in a horizontal mould. The shape of the piles was such as to allow them to be grooved into each other. The length varied, the longest being 52 ft. The size was 16 in. × 16 in. The 52 ft. piles each weighed about 6 tons, and when these were lifted to an angle of 35°, the deflection in the middle was only about ½ in.

The piles were driven by a steam pile driver with a ram weighing 2 tons, which struck a wooden dolly resting on the head of the pile.

Important reinforced concrete work has also been carried out in connection with the Manchester Ship Canal, the New York Harbour, also in harbour works at Southampton, and in Italy, Belgium, and Holland.

Reinforced Concrete Groynes.—The most modern method

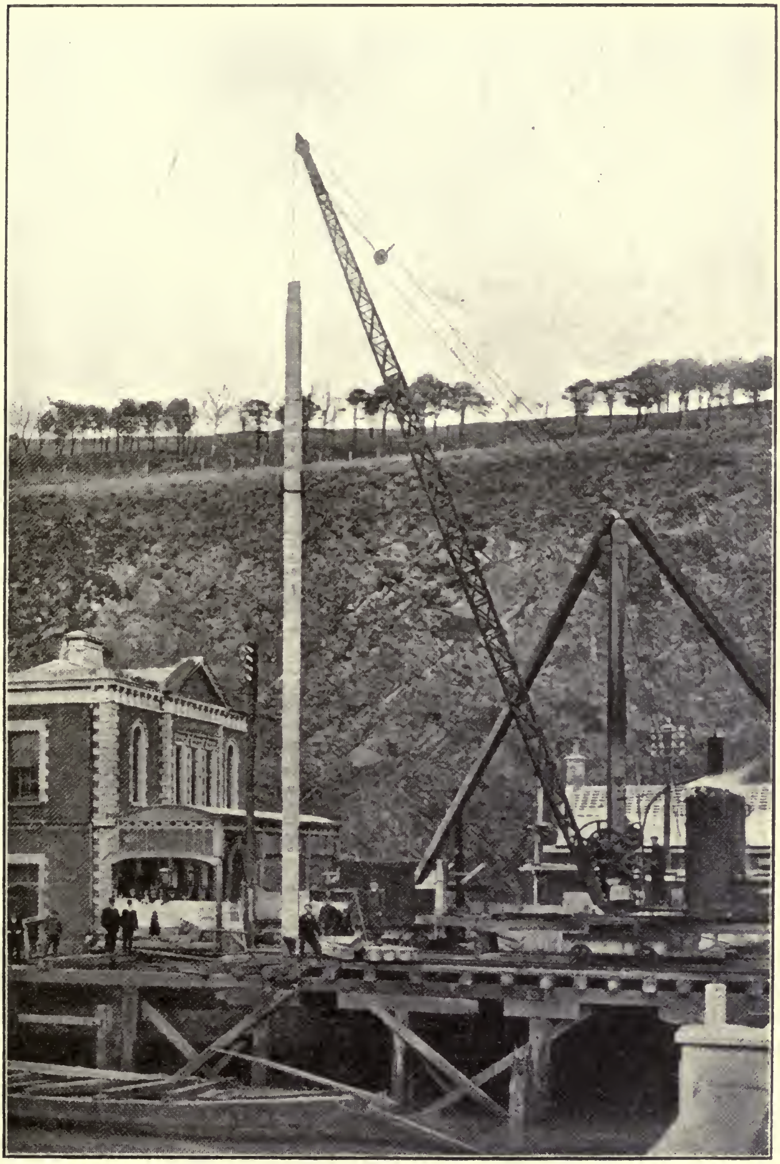


FIG. 221.

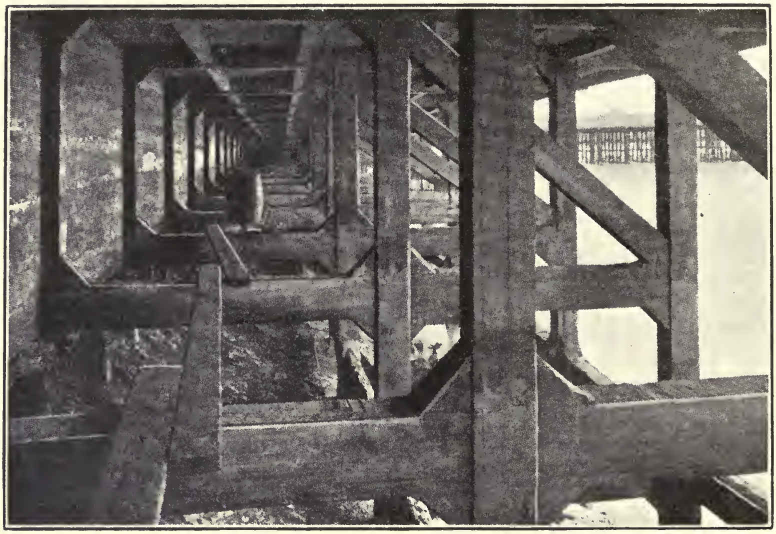


FIG. 222.

FIG. 223.

of groyne construction is undoubtedly to replace the timber piles used hitherto, with reinforced concrete piles. Messrs. Owens & Case are the patentees of groynes constructed in this way, and their system has been introduced in many places with great success.

Jetty Head.—A jetty head in 45 ft. of water has been constructed on the Considère system at Thames Haven for the London and Thames Haven Oil Wharves, Ltd.

The original head was constructed of timber, but was carried away by collision of a steamer.

FIG. 224.

The new head is 136 ft. long × 32 ft. wide, and stands 50 ft. from deck to river bed (see Figs. 225 and 226).

The work has been executed by the Considère Construction Co., Ltd.

Factories, Silos, Coal Bunkers.—Many of these have been erected in this material, almost every system of reinforcement having been employed.

Factory at Noisiel.—A large factory at Noisiel, near Paris, is in course of erection. Both the factory and the bridge approaching it are in reinforced concrete. The work has been carried out on the Considère system, the foundation piles used being the Considère spiralled piles. The bridge has a span of 146 ft., and the structures have been erected for the Menier Chocolate Co.

Over 500 spiralled piles were used in the foundations, the average length being 37 ft. The piles were octagonal, and 14 in. in diameter across flats.

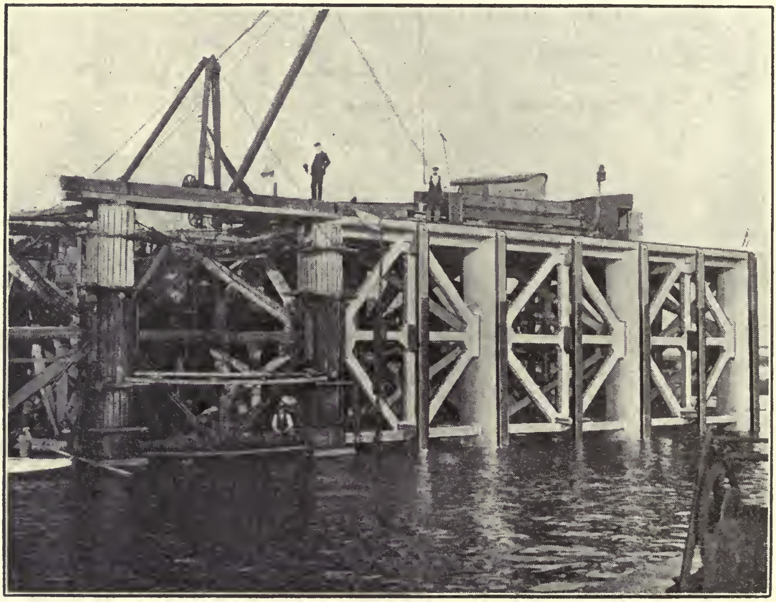


Fig. 225.

A 2 ton ram with a drop of 7 ft. was used for driving, the ram falling direct on to the head of the pile.

Each of the columns in the factory is 2 ft. 3 in. square.

Silos.—A very important building of this class is that designed by Messrs. Sir Alfred Gelder and L. Kitchen, FF.R.I.B.A. at Hull, for Messrs. Joseph Rank, Ltd. It is on the Wells system of reinforcement, and was built by Stuart's Granolithic Co., Ltd. (see Fig. 227).

It is one of the best examples of work of this class in this country.

The bins are practically self-contained, with the result that any combination of empty or full bins can be arranged.

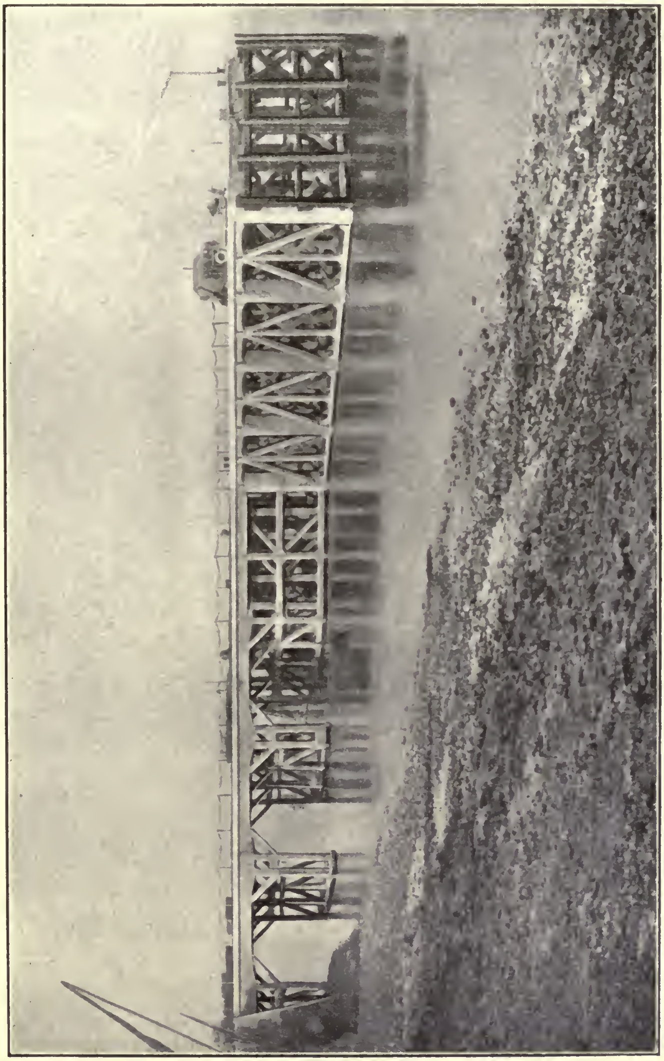


FIG. 226.

Coal Bunker at Walkmill Colliery.—This has recently been erected on the Moss system by Messrs. Wm. Moss & Sons, Ltd., It is fully illustrated in the drawing Fig. 228, and is a very interesting piece of engineering work. It is 95 ft. in length, inside measurement, 35 ft. above ground level to top of bunker, and 20 ft. in width, inside. The arrangement of the reinforcement is shown in the drawing.

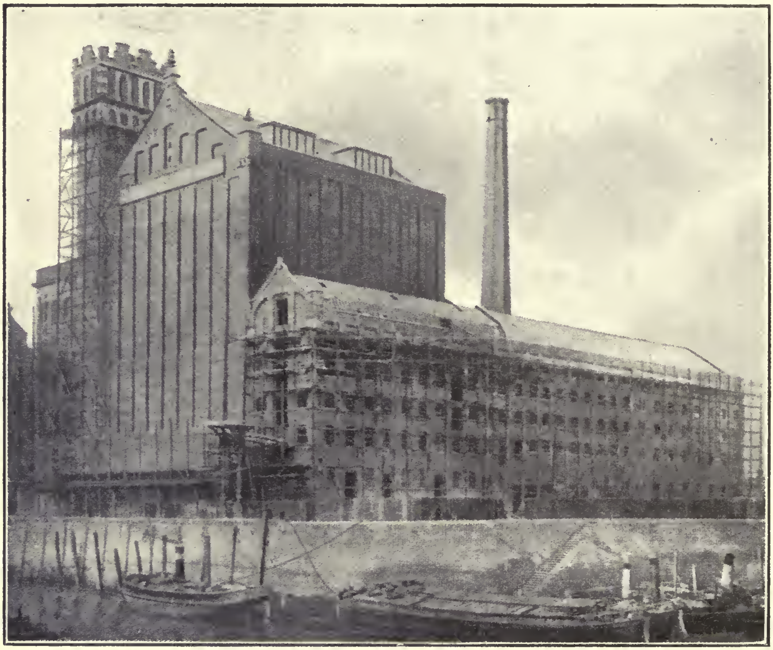


Fig. 227.

American Examples.—The best examples one can meet with of warehouse and factory construction in reinforced concrete are to be found in America, and the authors are able through the kindness of the engineers responsible for the designing of some of these buildings, to give illustrations of a few typical structures of this class.

Louisville and Nashville Railway Co's. Terminal Warehouse, Atlanta, Ga.—This is a five story building, 835 ft. long and 50 ft. wide. It is of reinforced concrete construction throughout, and is illustrated in Figs. 229-233.

Fig. 229 illustrates the construction of columns and beams.

Fig. 230 the structure in course of erection.

Fig. 231 a test load being applied to one of the floors.

Fig. 232 the arrangement of the reinforcement in the floors and beams.

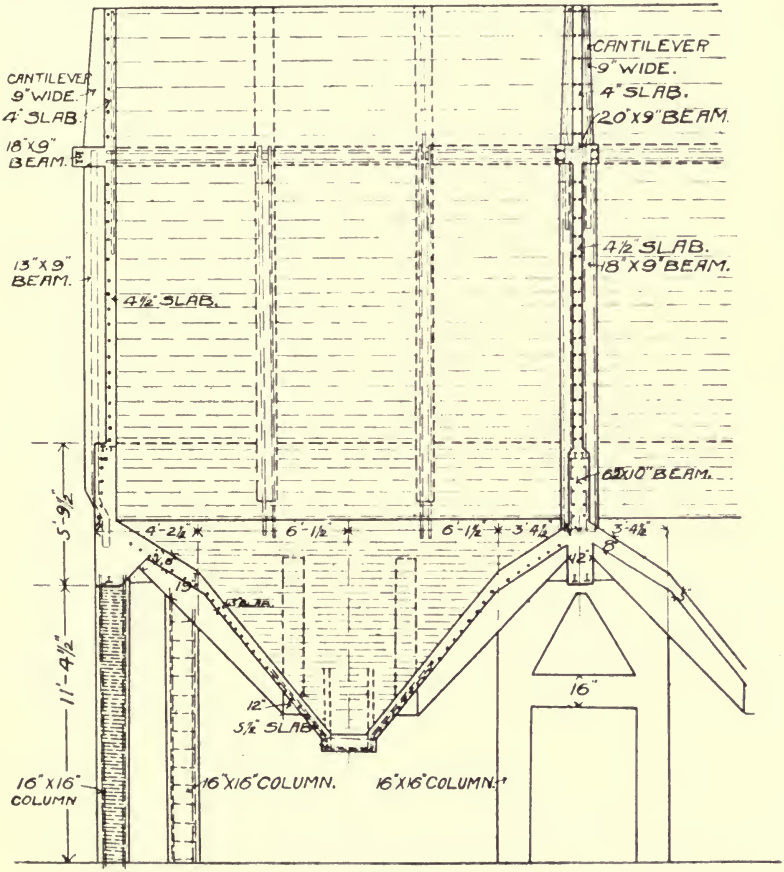


Fig. 228.

Fig. 233 a general view of the building.

It should be noted that the specifications required the floors to carry a test load of 1200 lb. per square foot. A skeleton construction was first erected, and reinforced concrete curtain walls were filled in afterwards. This interesting building is fully

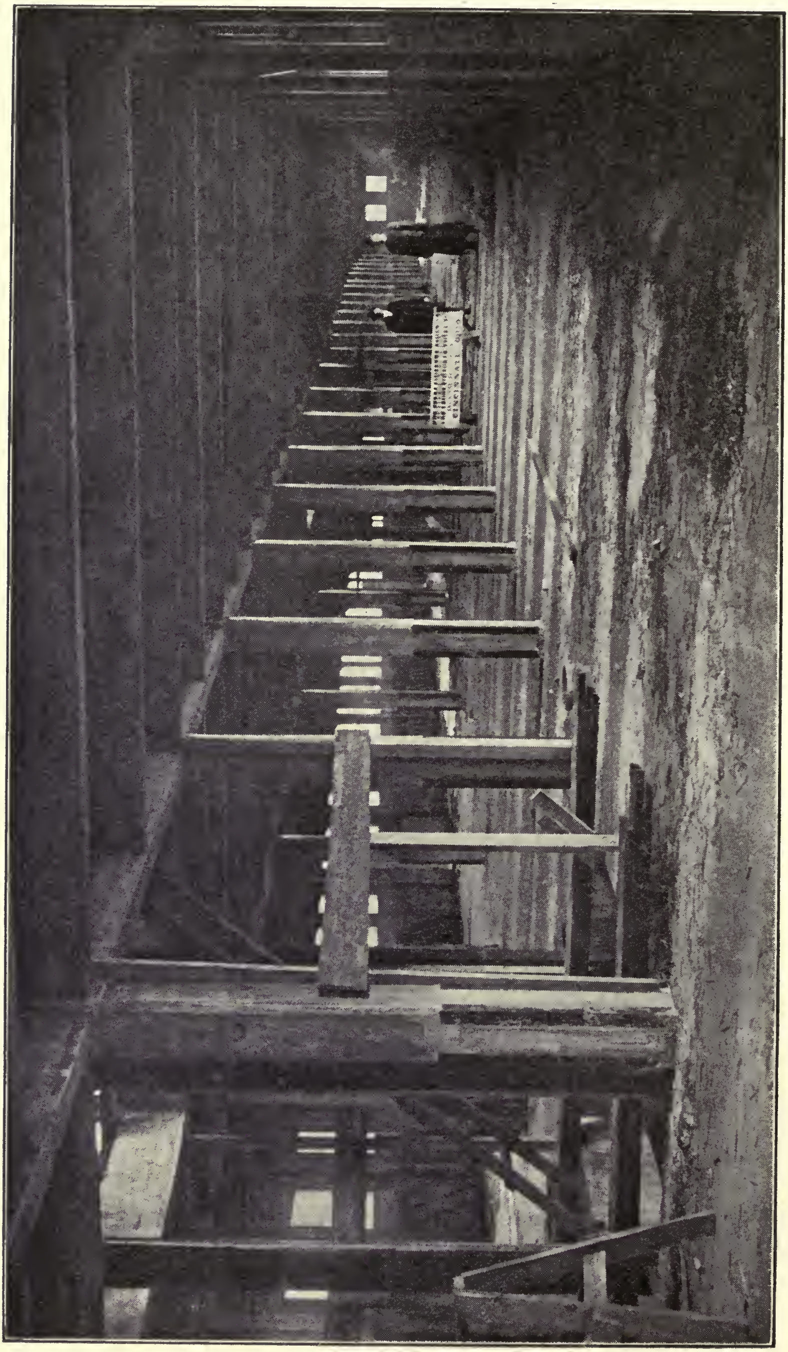


Fig. 229.

FIG. 230.

FIG. 231.

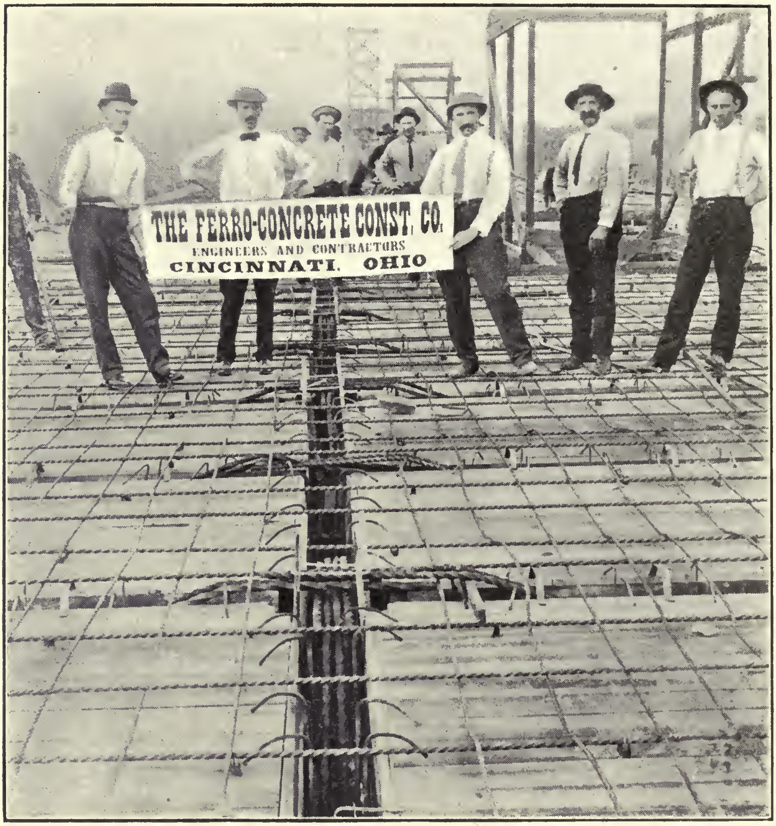


Fig. 232.

Fig. 233.

described in "The Railroad Herald" (of Atlanta), for January, 1907, and "The Manufacturers' Record," 3 January, 1907.

Ransome twisted steel bars were used as the reinforcement. The building was erected under the supervision of Mr. W. H. Courtenay, M.Am.Soc.C.E., the chief engineer to the L. and N.R.R. Co., Louisville, Ky., the work being executed by the Ferro-Concrete Construction Co., of Cincinnati.

Figs. 234, 235 represent a machine shop built for the Taylor-Wilson Manufacturing Co., McKees Rocks, Penn.; Engineer, Mr. Robt. A. Cummings, M.Am.Soc.C.E.

Fig. 234.

Fig. 236 shows the main building of the Minterburn Mills Co., Rockville, Conn.

Dimensions, 58 ft. by 294 ft.

Engineers, Messrs. C. R. Makepeace & Co.

Builder, Mr. F. B. Gilbreth.

Fig. 237 illustrates the carpentry shop of the National Cash Register Co., Dayton, Ohio.

The reinforcement was designed by The General Fireproofing Co., the contractors being the Expanded Metal Fireproofing Co.

Fig. 235.

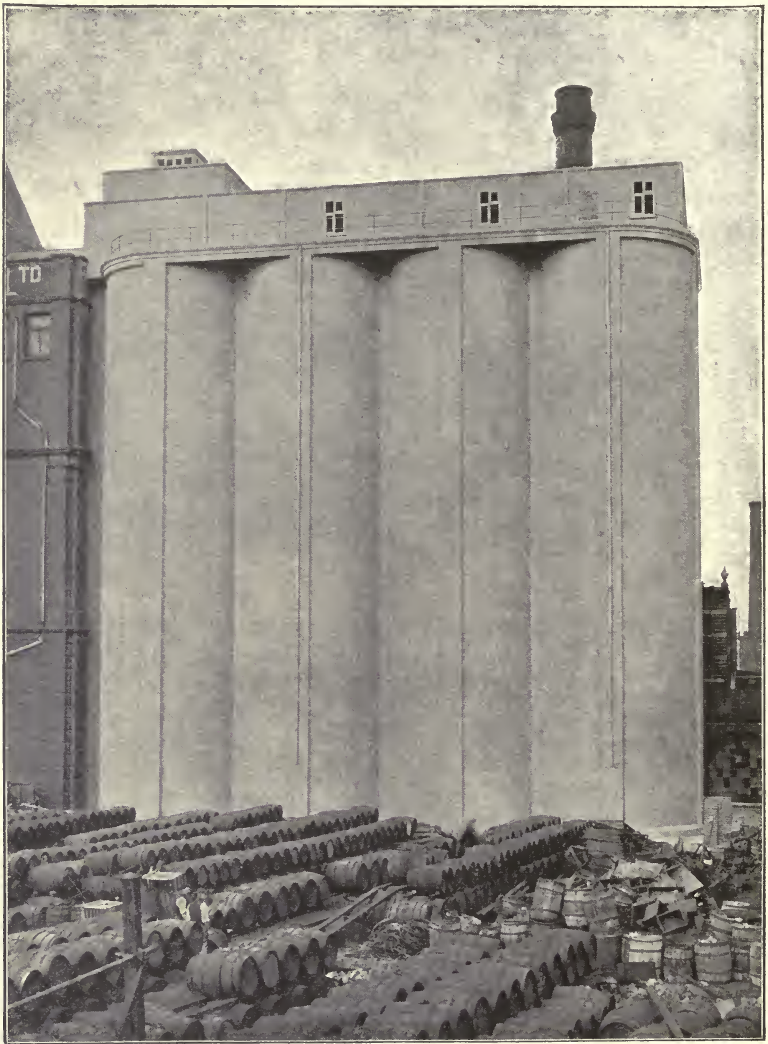
Fig. 236.

Fig. 238 represents the Smiths' and Boiler Shop of the Atlas Portland Cement Co. This was built in 1906. The shop is 309 ft. 9 in. long by 55 ft. 6 in. wide by 31 ft. high to eaves.

Fig. 237.

Fig. 238.

Grain Silos at Silvertown, England.—Some important grain silos have recently been constructed on the Hennebique system at Silvertown, for the Co-operative Wholesale Society. There are in all eight cylindrical silos, each 80 ft. high by 20 ft. diameter, built upon Hennebique piles. The building also contains



"*Concrete and Constructional Engineering.*"

Fig. 239.

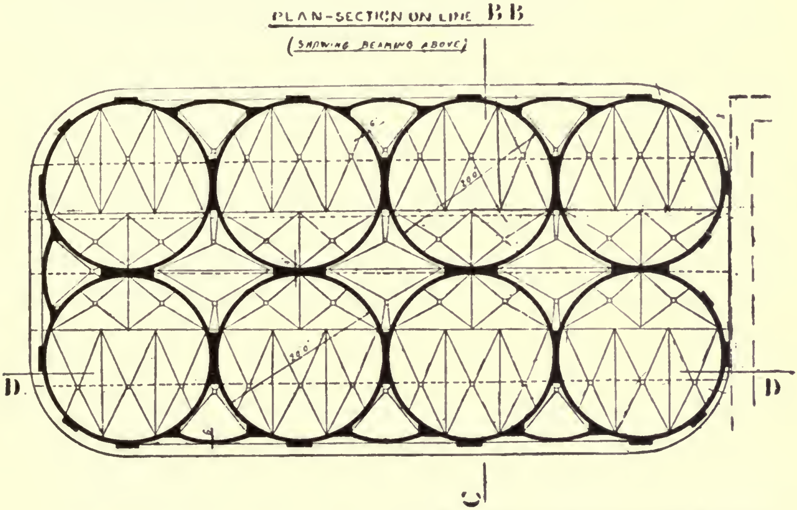
a number of tunnels and subways all built in reinforced concrete.

The dimensions of the building are as follows: length 85 ft. 6 in., width 43 ft. 6 in., height above ground level 102 ft. There is a storage capacity in each silo of 505 tons, or 2242 quarters of grain, the total capacity being approximately 18,000 quarters.

The walls of these great bins are only 6 inches in thickness, the saving of space by the adoption of reinforced concrete is therefore very considerable.

Fig. 239 represents a view of four of these silos.

Fig. 240 shows a sectional plan of the silos.



" *Concrete and Constructional Engineering.*"

FIG. 240.

This interesting structure has been fully described in " Concrete and Constructional Engineering," November, 1910; it was designed by Mr. F. E. L. Harris, A.R.I.B.A., of Manchester, the contractors being Messrs A. Jackaman & Son.

The foundations are supported by 124 Hennebique piles, 14 in. by 14 in., varying in length from 27 to 35 ft.

Raw Meal Silos near Rugby.—The raw meal silos at Messrs. Kaye & Co.'s Cement Works, Southam, Rugby, are constructed in reinforced concrete; they are illustrated in Figs. 241, 242, 243, and were designed by Mr. W. Gilbert, M.Inst.C.E., the work being carried out by the Expanded Metal Co., Ltd.

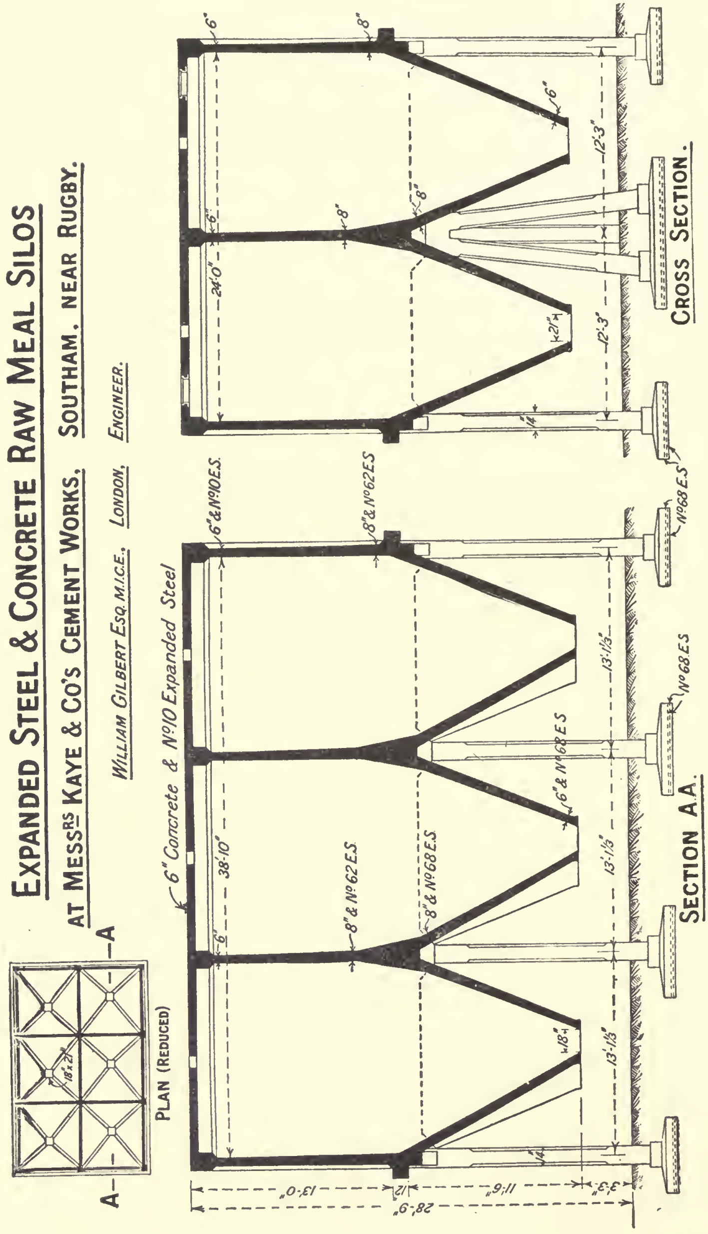

EXPANDED STEEL & CONCRETE RAW MEAL SILOS
AT MESSRS KAYE & CO'S CEMENT WORKS, SOUTHAM, NEAR RUGBY.
WILLIAM GILBERT ESQ. M.I.C.E., LONDON, ENGINEER.
PLAN (REDUCED)
A
A
18"x21"
6" Concrete & No10 Expanded Steel
6" & No10 E.S.
8" & No62 E.S
8" & No62 E.S
8" & No68 E.S
6" & No68 E.S
No68 E.S
No68 E.S
38'10"
24'0"
12'3"
12'3"
13'1½"
13'1½"
13'1½"
13'0"
11'6"
3'3"
28'9"
CROSS SECTION.
SECTION A.A.

FIG. 241.

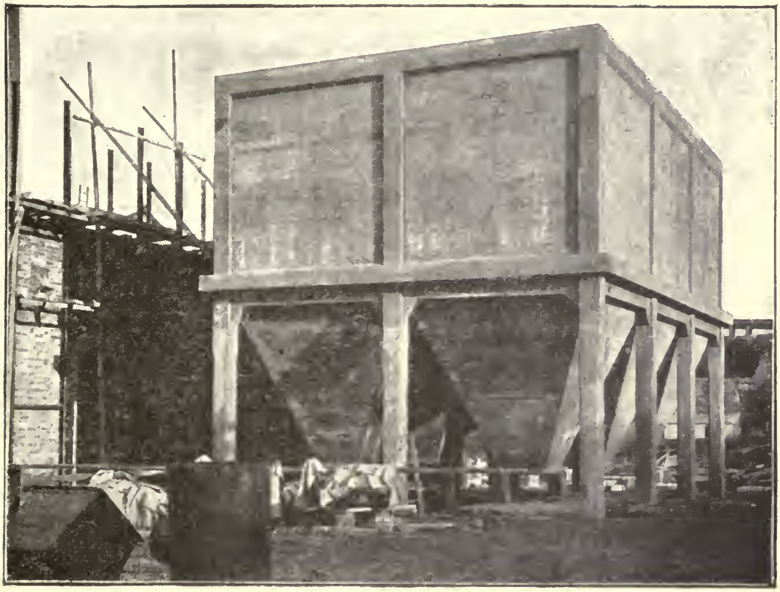


Fig. 242.

Fig. 243.

The reinforcement consists chiefly of 3 in. mesh expanded metal, with some indented bars and plain bars. Expanded metal was used in the column footings, bins and roof, indented bars in the columns, and plain bars in the bins.

The aggregate used for the concrete was crushed granite. There are six bins, each 12 ft. 6 in. by 11 ft. 6 in. inside measurements; each bin has a carrying capacity of 60 tons; the complete structure being 40 ft. 6 in. by 26 ft. by 30 ft. above ground level. The columns are 14 in. by 14 in., the outside ones resting on footings 5 ft. by 5 ft. by 9 in. thick reinforced by the insertion of No. 68 expanded metal near the underside, and the two inside pairs of columns rest on footings 10 ft. by 6 ft. by 9 in. thick, with two layers of No. 68 expanded metal near the underside, and one layer between the columns near the topside.

The walls of the silos are 8 in. thick, tapering to 6 in. at the mouth, and are reinforced with one layer of No. 68 expanded metal and with round bars; the bars are threaded and left projecting beyond the concrete to carry cast iron mouthpieces, the outlet being 1 ft. 9 in. by 1 ft. 6 in.

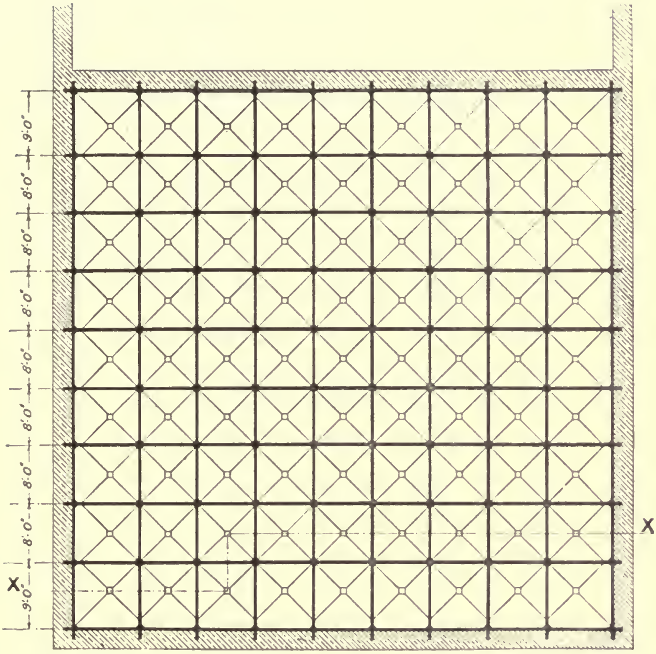
The roof is 6 in. thick, and is reinforced with No. 10 expanded metal, and over each bin is inserted a manhole frame and cover for inspection purposes, a filling hole 10 in. by 10 in. being left at the centre of each bin.

Typical Example of Grain Silos.—Figs. 244, 245, 246 represent plan and section of typical grain silos.

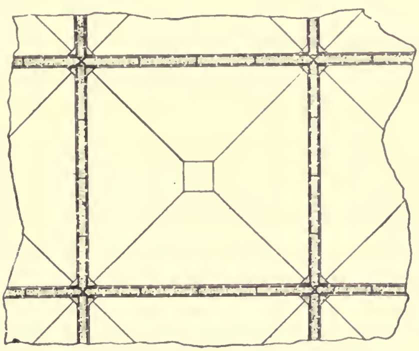
Coal Pocket.—Fig. 247 represents a coal pocket constructed in reinforced concrete at Messrs. Tate's Sugar Works, London. The reinforcing steel used was expanded metal.

Coke Hoppers.—The reinforced concrete coke hoppers and water tank illustrated in Figs. 248, 249, 250 were designed by Mr. S. R. Ogden, the works being executed by the Expanded Metal Co., Ltd. The overall dimensions of the structure are 48 ft. by 29 ft., the height from ground to top of tank being 46 ft. 6 in.

The reinforcement consisted of expanded metal and plain bars, with the addition of a few trussed and indented bars. The total weight of the structure in use is about 280 tons, the

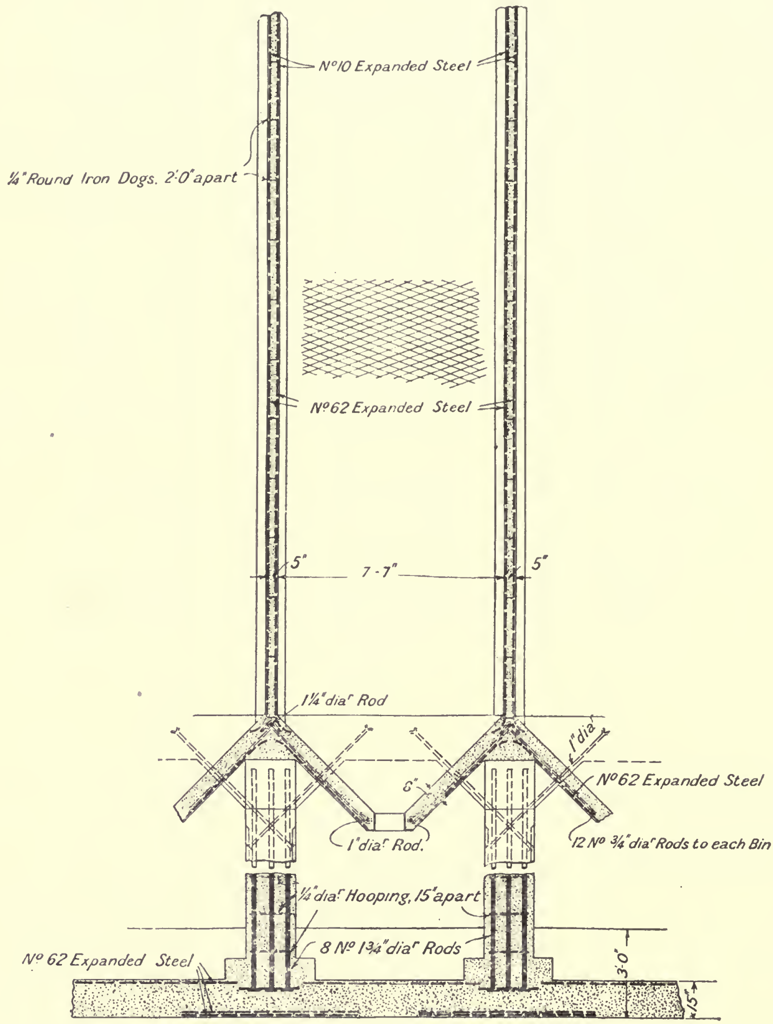


PLAN OF GRAIN SILOS.



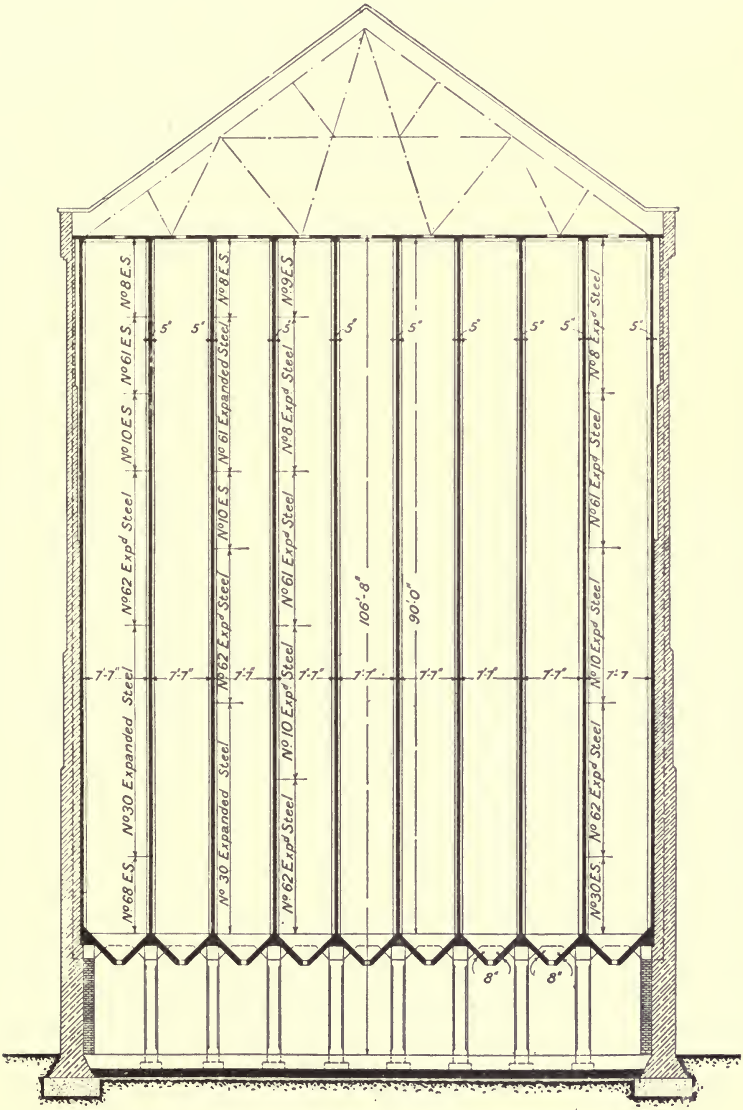
ENLARGED PLAN OF ONE BIN.

FIG. 244.



GRAIN SILOS. — ENLARGED SECTION OF ONE BIN.

FIG. 245.

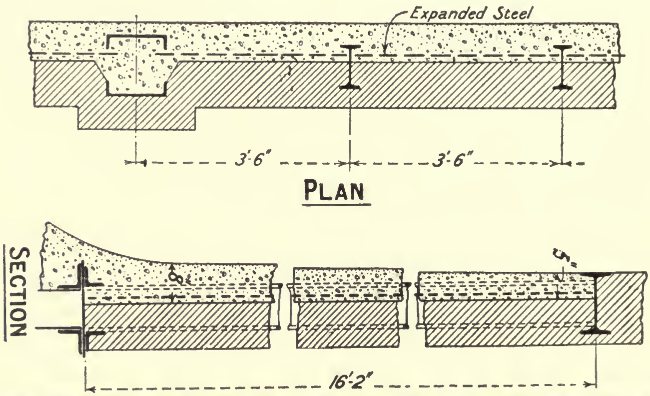


GRAIN SILOS.

Fig. 246.

hoppers, when full, carrying about 90 tons of coke, the capacity of the water tank being about 100 tons.

The centre columns vary in size from 20 in. by 20 in. to



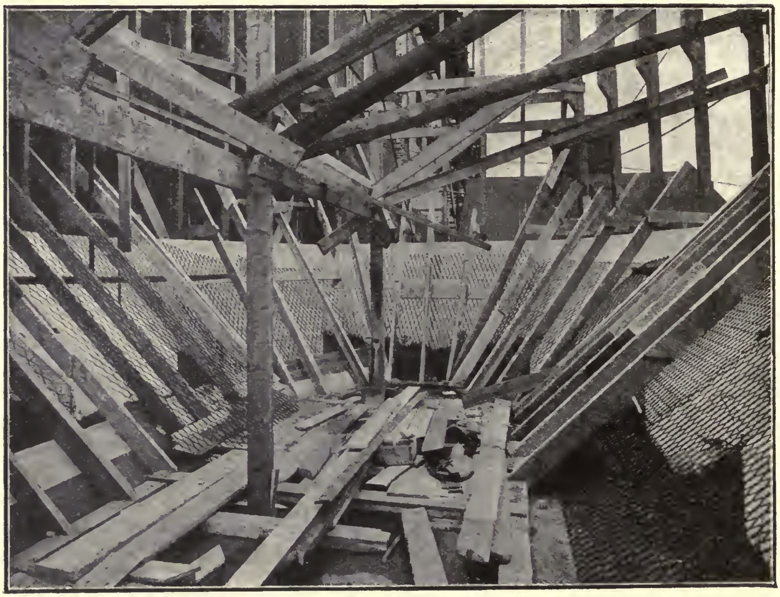


Fig. 247.

16½ in. square; the side columns being 15 in. square. The bases for the columns vary in size from 5 ft. square to 8 ft. 3 in. square.

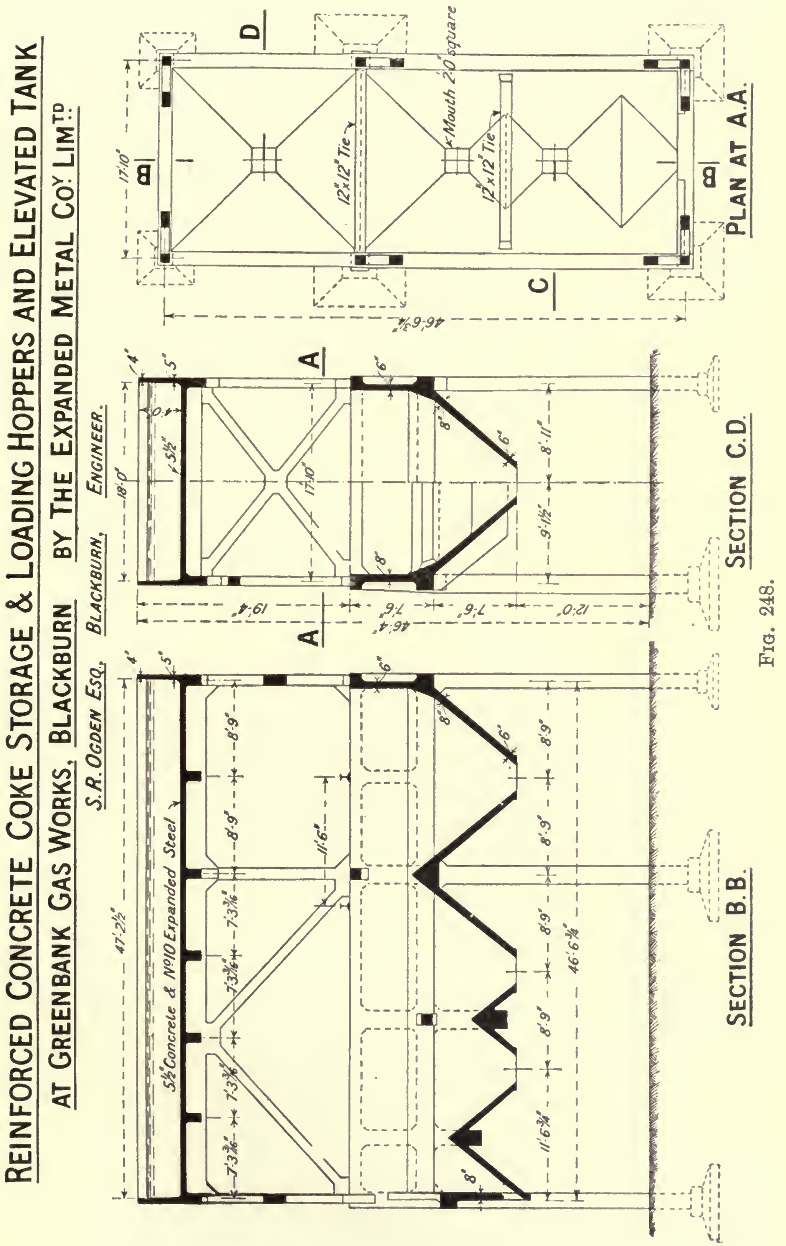

REINFORCED CONCRETE COKE STORAGE & LOADING HOPPERS AND ELEVATED TANK
AT GREENBANK GAS WORKS, BLACKBURN BY THE EXPANDED METAL COY. LIMTD.
S.R. OGDEN ESQ. BLACKBURN. ENGINEER.
SECTION B.B.
SECTION C.D.
PLAN AT A.A.
5½" Concrete & No10 Expanded Steel
12"x12" Tie
Mouth 2'0" square
47'2½"
46'6¾"
46'4"
17'10"
18'0"
19'4"
7'6"
12'0"
8'11"
9'1½"
8'9"
11'6"
11'6¾"
7'3¾"
A
B
C
D

FIG. 248.

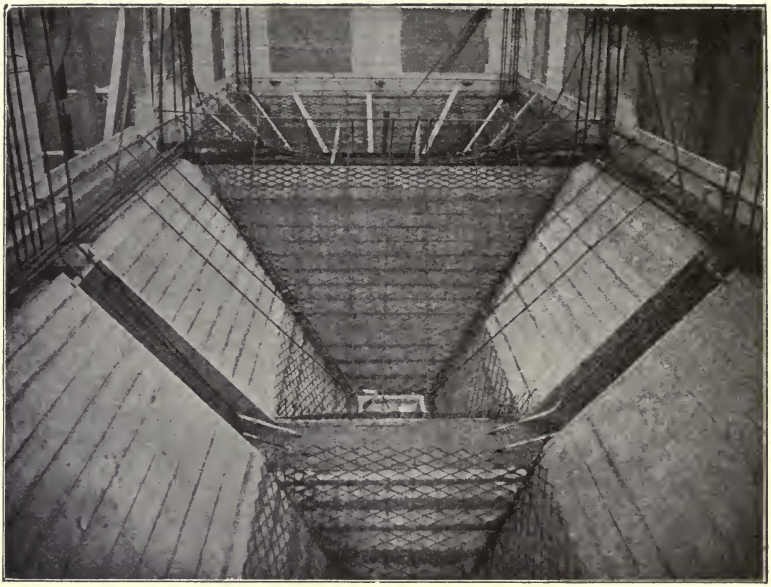


Fig. 249.

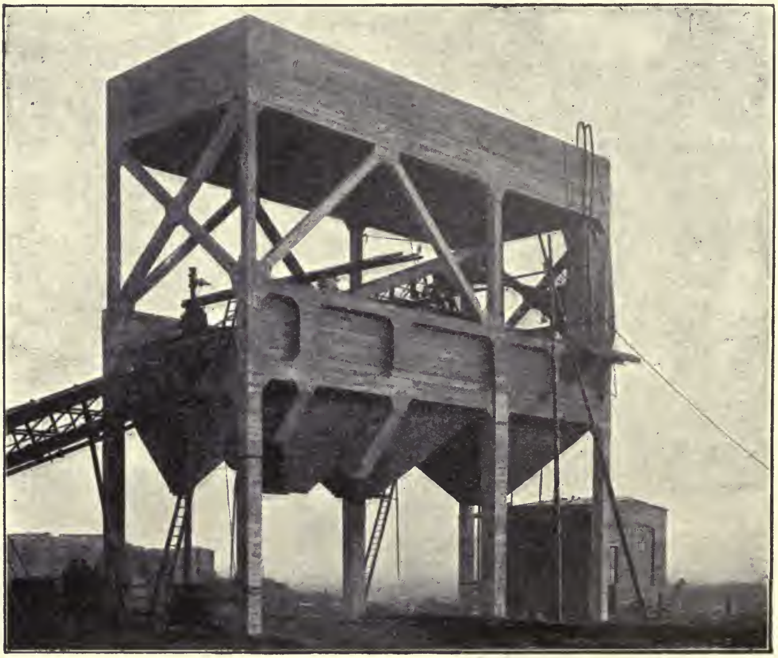


Fig. 250.

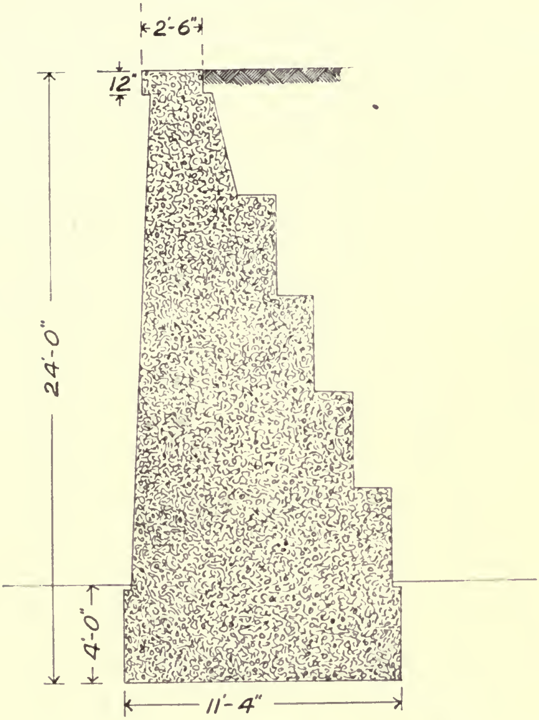


FIG. 251.

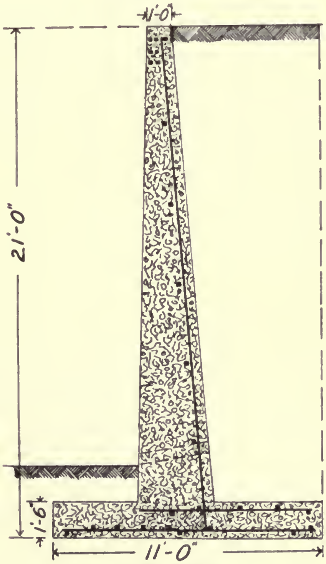


FIG. 252.

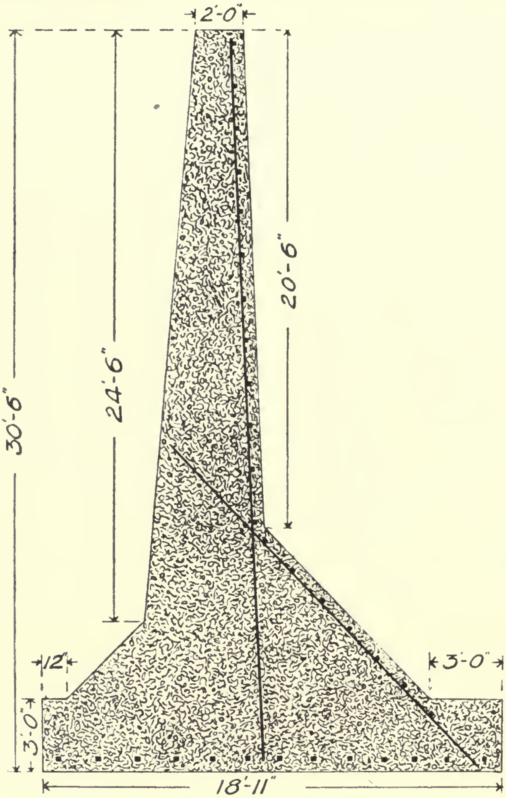


FIG. 253.

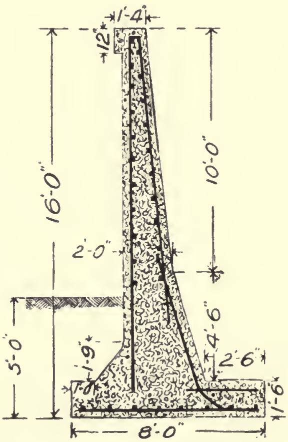


FIG. 254.

Retaining Walls and Boundary Walls.—The material has been used very extensively for this purpose both in this country and in America.

American Examples.—In a paper which appeared in the August, 1910, "Transactions of the Society of Engineers,"[1] the author describes briefly one or two important reinforced con-

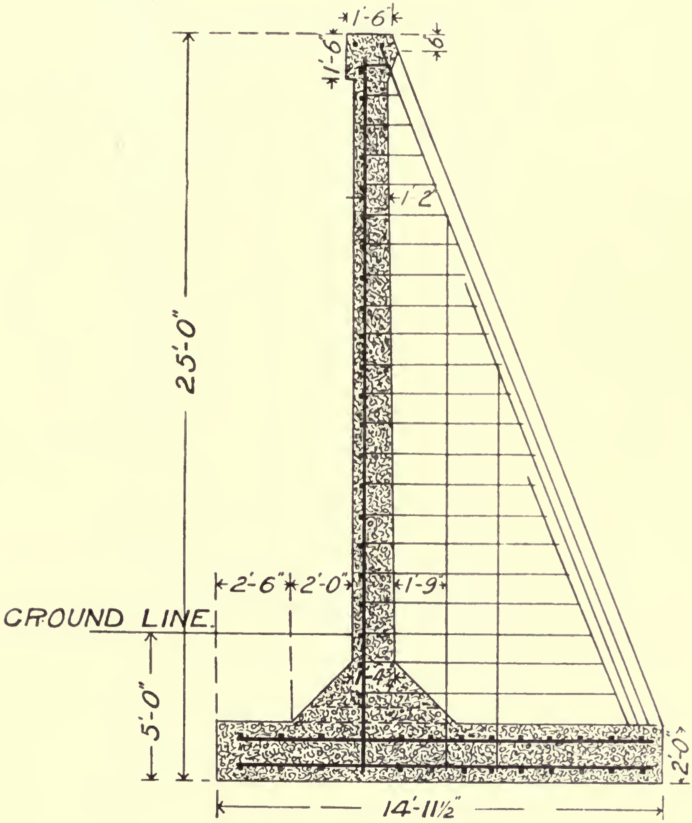


Fig. 255.

crete retaining walls, American examples. Two of these are illustrated in Figs. 257, 258.

They represent very large hollow retaining walls constructed

[1] "Reinforced Concrete Retaining Walls," by E. R. Matthews. "Jour. Soc. of Engineers," August, 1910.

in reinforced concrete on the Atlanta, Birmingham and Atlantic Railroad. These walls were designed by Mr. Alex. Bonnyman, chief engineer, who writes regarding them:—

"By the contract prices for plain concrete and reinforced concrete, we figure that there is a saving in the reinforced walls of about 25 per cent."

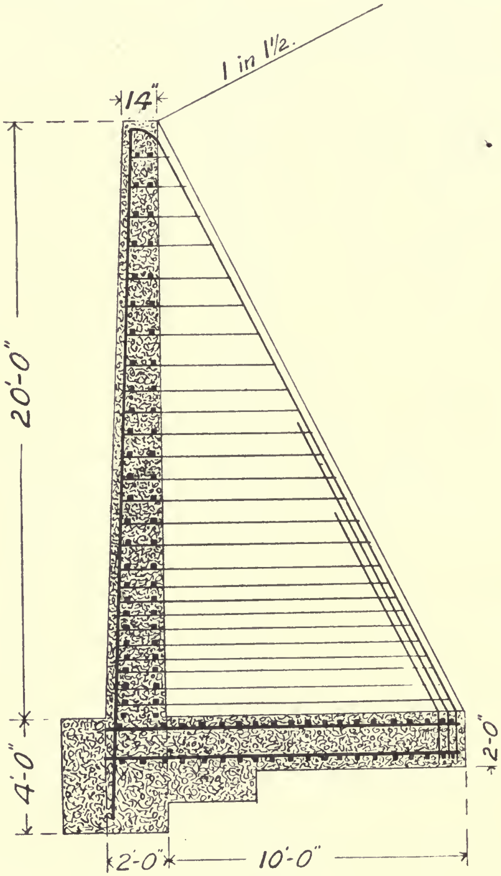


Fig. 256.

The wall shown in Fig. 257 is 61 ft. in height.

English Examples.—Some examples have already been given by the authors in the chapter on Municipal Engineering. Others will now be given.

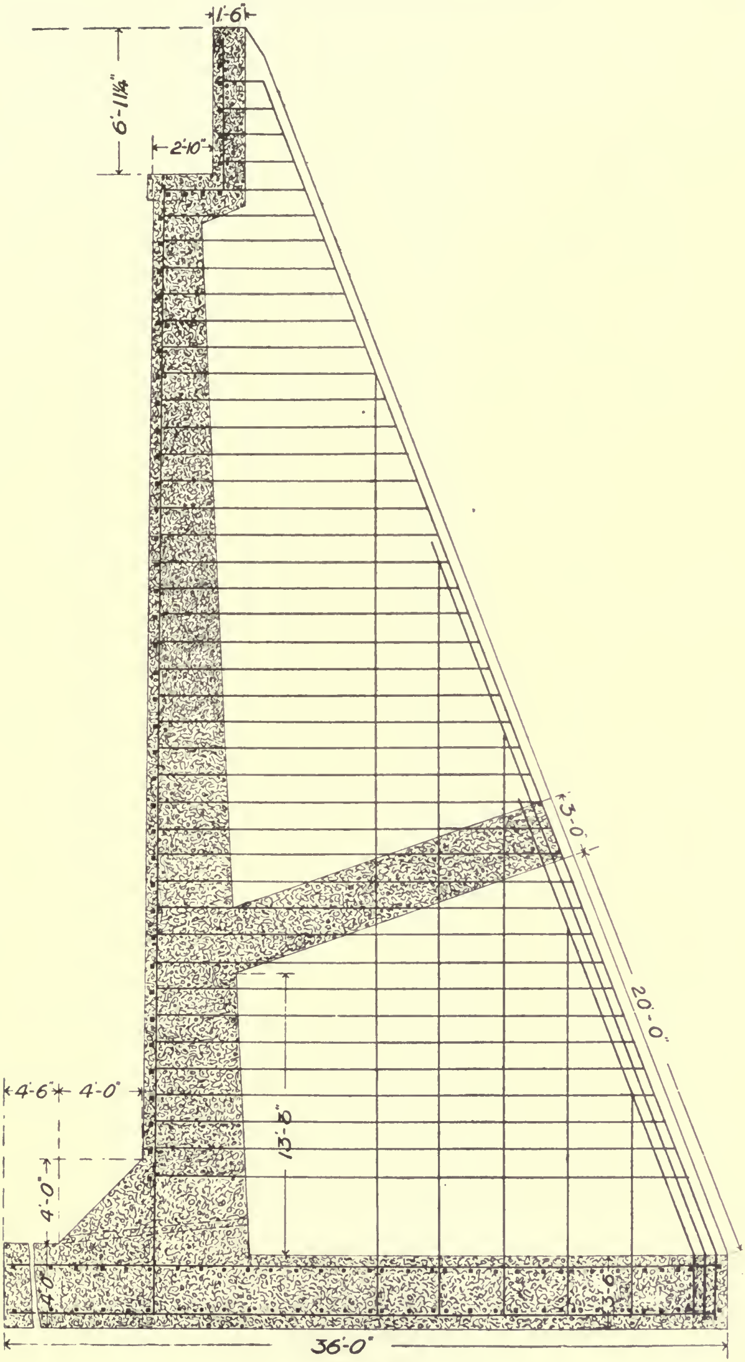


Fig. 257.

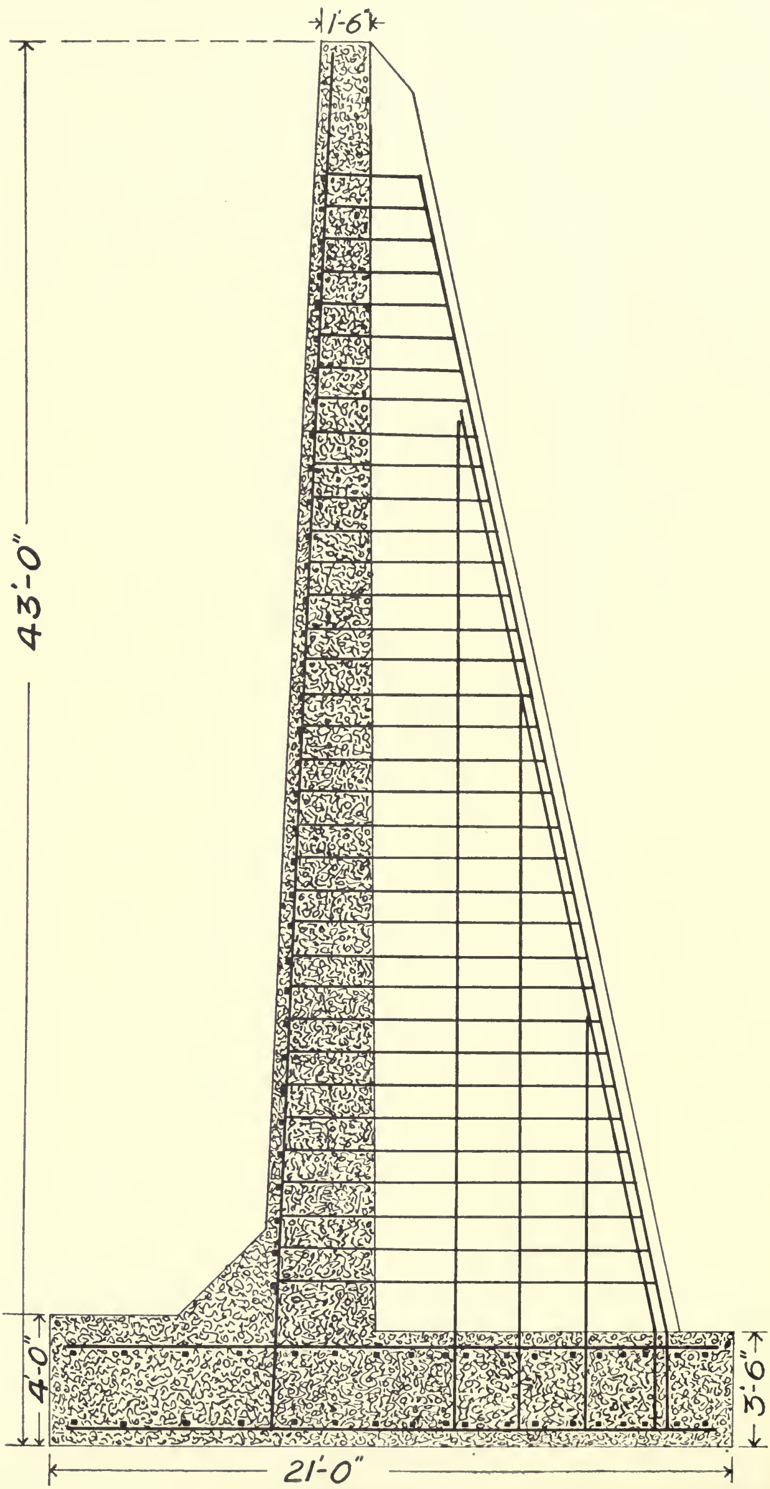


Fig. 258.

Boundary Wall, West Hartlepool Cemetery.—This is constructed in reinforced concrete. It was designed by the Borough

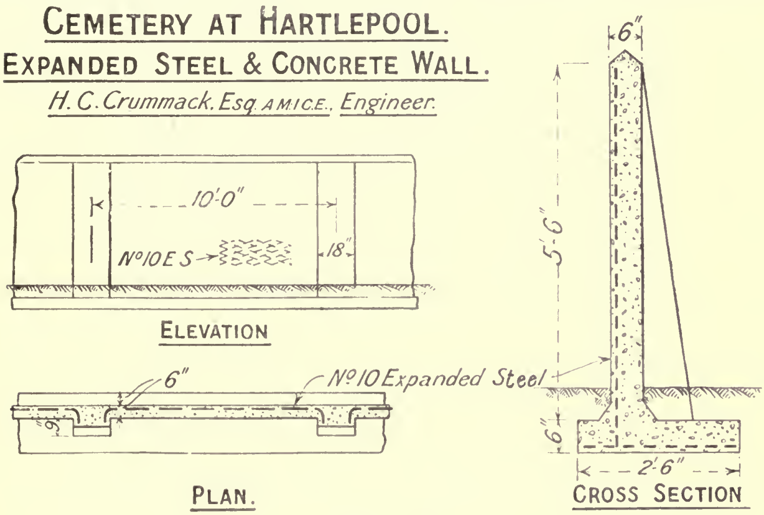


FIG. 259.

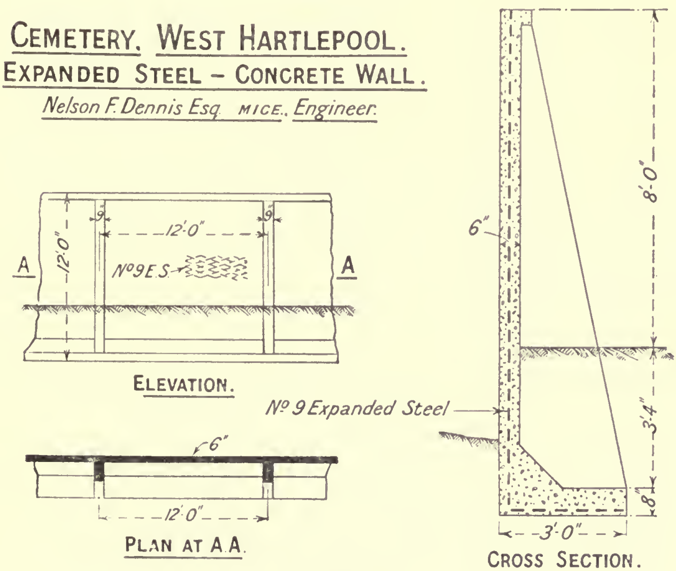


FIG. 260.

Engineer, Mr. Nelson F. Dennis, A.M.I.C.E., and is illustrated in Figs. 259-262.

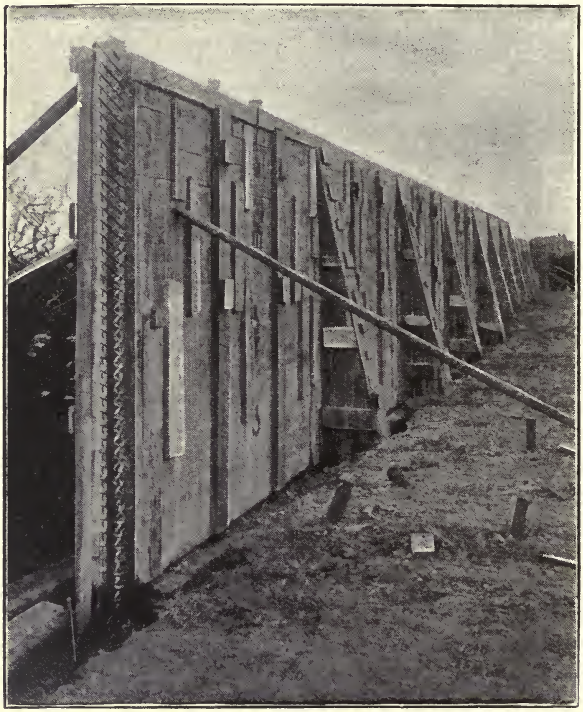


Fig. 261.

Fig. 262.

Cross-sections through the wall are shown in Figs. 259 and 260.

The reinforcement used was expanded metal.

Retaining Wall at Birkenhead Gas Works.—This wall was designed by the engineer, Mr. T. O. Paterson, M.Inst.C.E., and expanded metal was used as the reinforcement. The wall is 159 ft. in length, and 23 ft. in height above ground level. This is an interesting piece of reinforced concrete work.

Retaining Wall at Guildford.—This wall is carried on piles and is reinforced with expanded metal and plain bars. Buttresses

Fig. 263.

are built at 5 ft. 6 in. centres, these are 9 in. in thickness. The wall tapers from 9 to 6 in. in thickness, and was designed by the Borough Engineer, Mr. G. H. Mason, A.M.I.C.E.

The Patent Indented Steel Bar Co. have done some excellent work of this class, a few examples of which are now given :—

Wall for Coal Store at Hereford Gas Works.—This wall acts as a cantilever attached to a horizontal slab, without the assistance of counterforts. The wall was designed by the Indented Steel Bar Co., Ltd., and encloses a coal store. It is 10 ft. 6 in. high and 150 ft. in length, and is illustrated in Fig. 263, which shows the arrangement of the reinforcement. The roof principals are carried by buttresses spaced at regular intervals.

Wall at Royal Insurance Offices, Piccadilly.—This wall was designed by the Indented Bar Co., and built under the supervision of Mr Alex. Drew, M.I.Mech.E. The additional area gained by building a reinforced concrete wall in a case of this kind may be gathered by glancing at the section shown in Fig. 264.

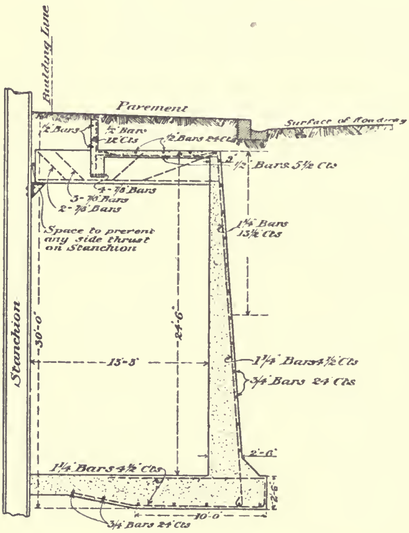


Fig. 264.

If this wall had been of mass concrete it would have been about 9 ft. thick at the base, whereas it is only 2 ft. 6 in. This wall was constructed in 1908, it is 24 ft. 6 in. in height, 9 in. in thickness at the top, and supports the heavy traffic of Piccadilly.

Other illustrations might have been given, but space will not permit.

Reinforced Concrete Stands.

Stand at York Racecourse.—This is an interesting piece of reinforced work; it has been carried out on the Kahn system, and is illustrated by the accompanying drawings, and two photos. The structure consists of an approach, bridge, and stand.

The approach road is carried by reinforced columns 10 in. by 10 in. reinforced by four ¾ in. bars running from base to top of parapet. The bases for the columns are 2 ft. 6 in. by 2 ft. 6 in. by 6 in. thick, and are reinforced by six ¾ in. bars.

The parapet walls are 5 in. in thickness; one parapet is 5 ft. 6 in. high, the other 4 ft.; these walls are reinforced by $\frac{3}{4}$ in.

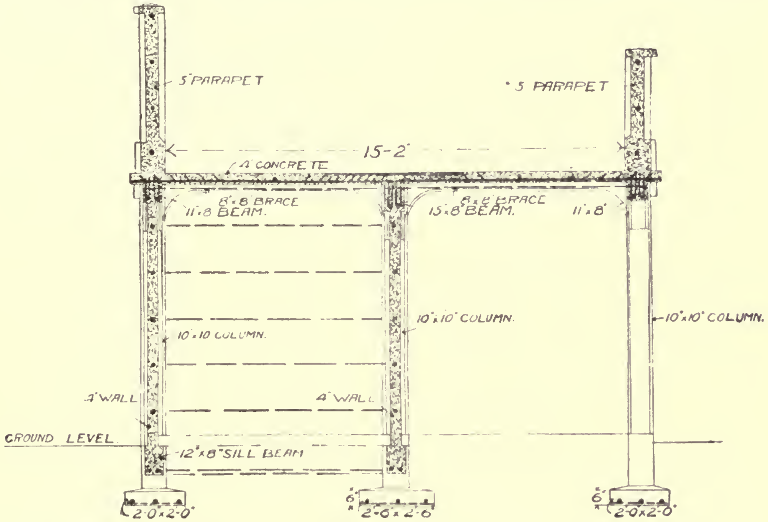


FIG. 265.

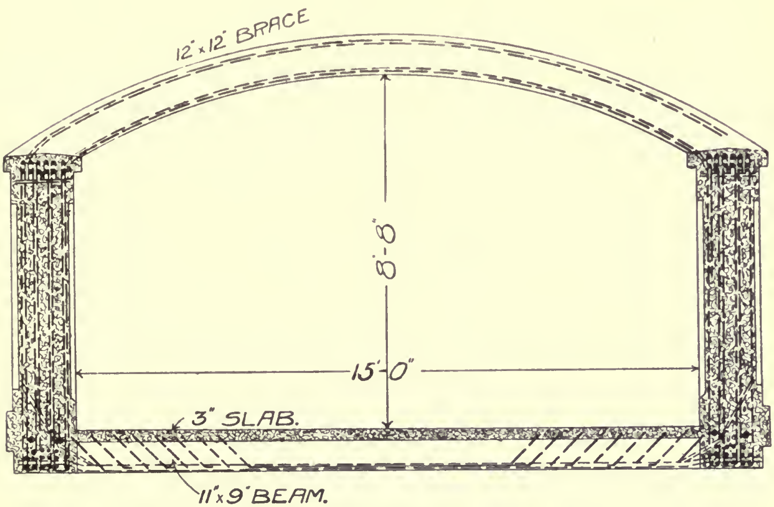


FIG. 265A.

bars. The roadway is formed of reinforced concrete, 4 in. in thickness, the reinforcement consisting of $\frac{1}{2}$ in. and $\frac{3}{4}$ in. bars.

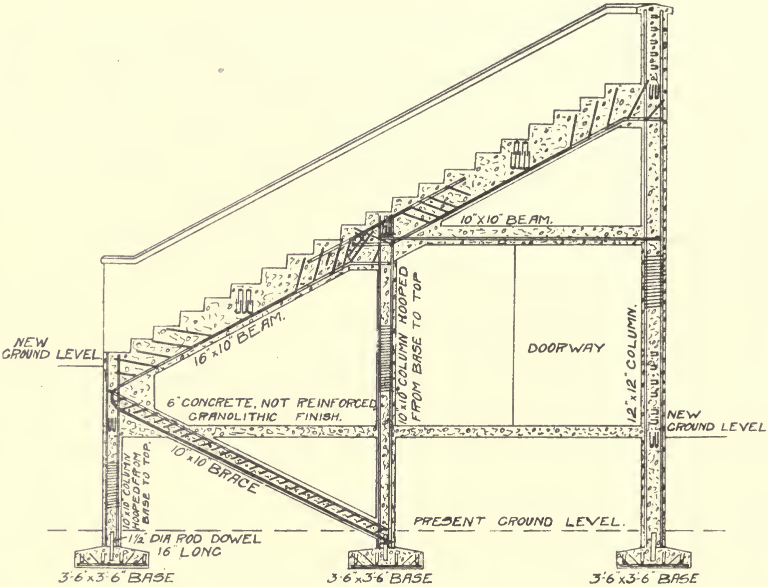


Fig. 266.

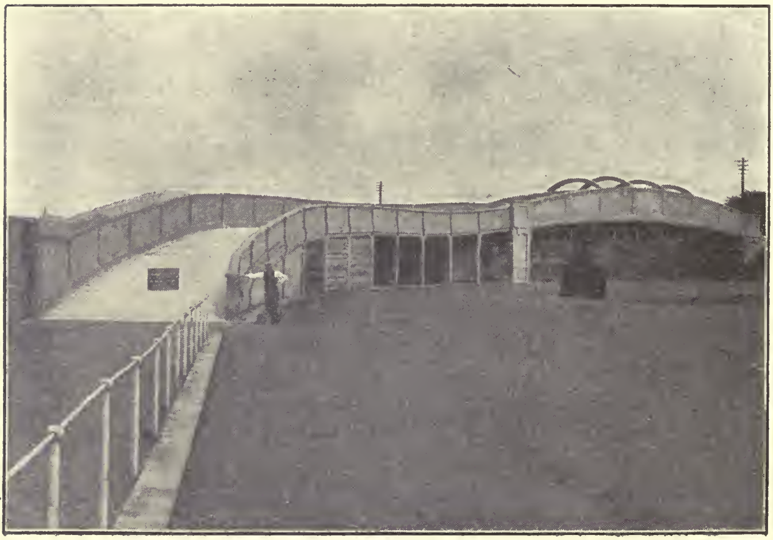


Fig. 267.

The approach road is 15 ft. 2 in. in width between parapets. The bridge has a clear span of 74 ft. 9 in., with a 13 ft. 2½ in. headway. The height of the top of the bridge above ground level is 21 ft. 2½ in. The whole structure is of reinforced concrete.

The stand is an interesting piece of work, the beams, decking, ties, and supports being shown in Figs. 265 and 266.

Reinforced Concrete Stadium.—The reinforced concrete stadium at the Franco-British Exhibition is another excellent example of work of this class; this was described in "Concrete

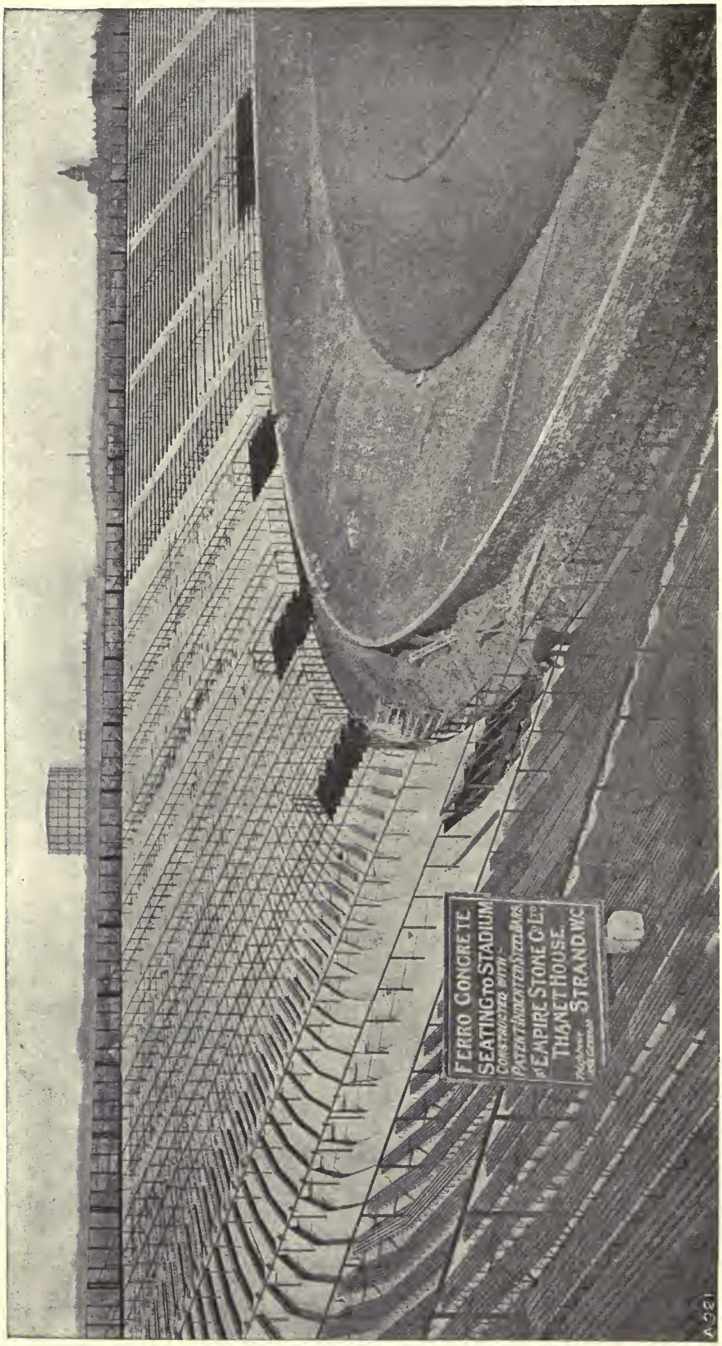
Fig. 268.

and Constructional Engineering," May, 1908. The structure is illustrated in Fig. 269.

Reinforced Concrete Football Stand at Bradford.—This is another good example of the use of this material in the construction of stands. The structure is illustrated in Fig. 270.

The architect for this work was Mr. Archibald Leitch, M.I.Mech.E., and the contractors were Messrs. John Ellis and Sons, Ltd.; the design of the structural members of the reinforced concrete work and of the steel reinforcement were prepared by Messrs. F. A. Macdonald and Partners of Glasgow.

Reinforced Concrete Stadium at Syracuse, U.S.A.—This stadium has recently been built at the Syracuse University. It



"*Concrete and Constructional Engineering.*"

FIG. 269.

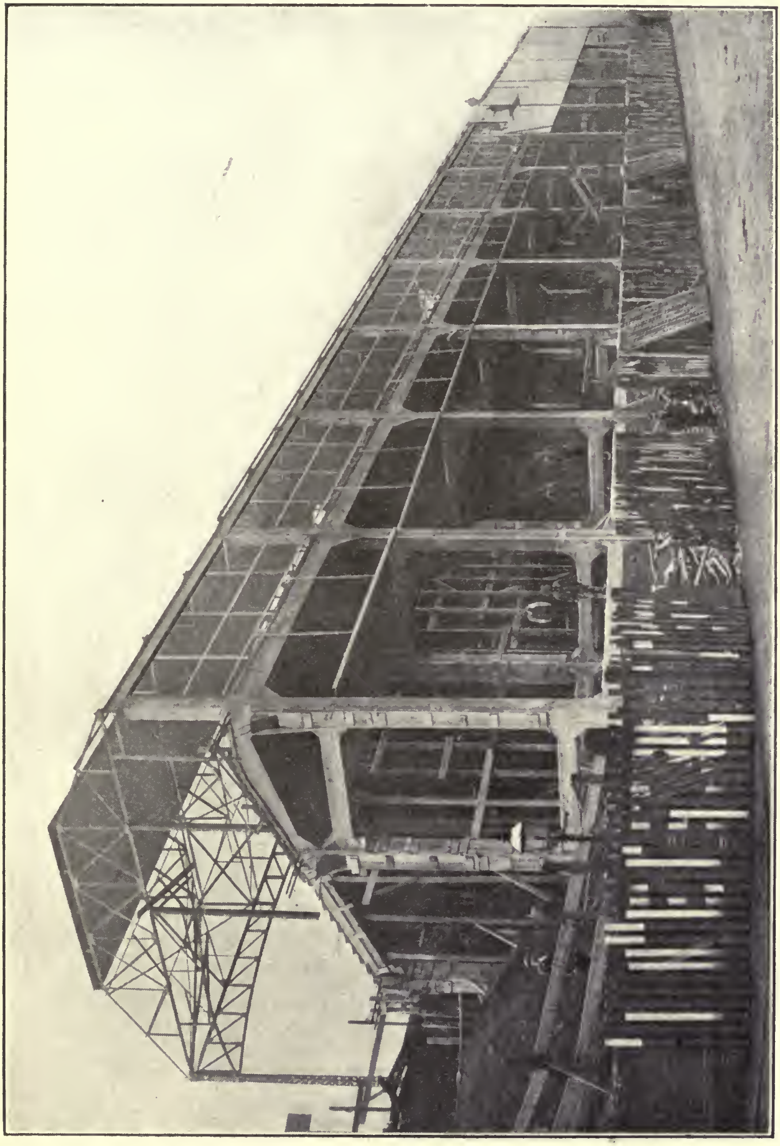


Fig. 270.

"*Concrete and Constructional Engineering.*"

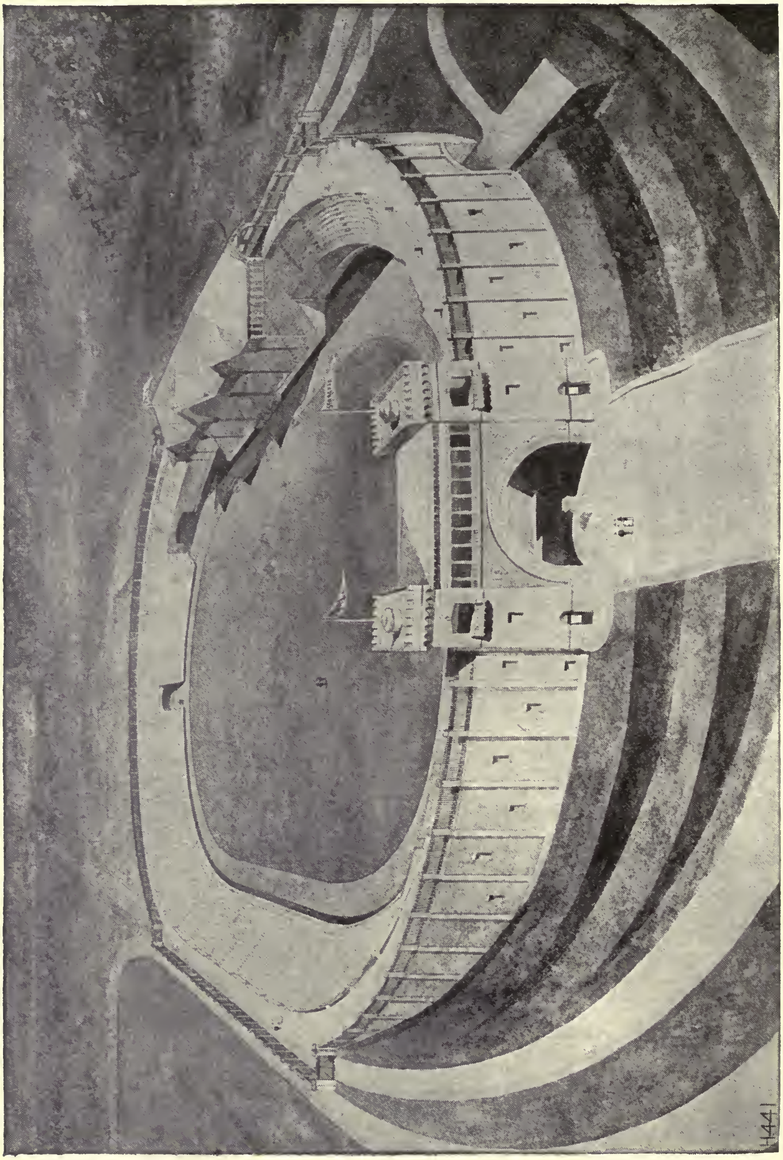


Fig. 271.

"Concrete and Constructional Engineering."

is 670 ft. in length, and covers an area of 6⅓ acres. It has a normal seating capacity for 20,000 with a possible seating capacity of nearly double this.

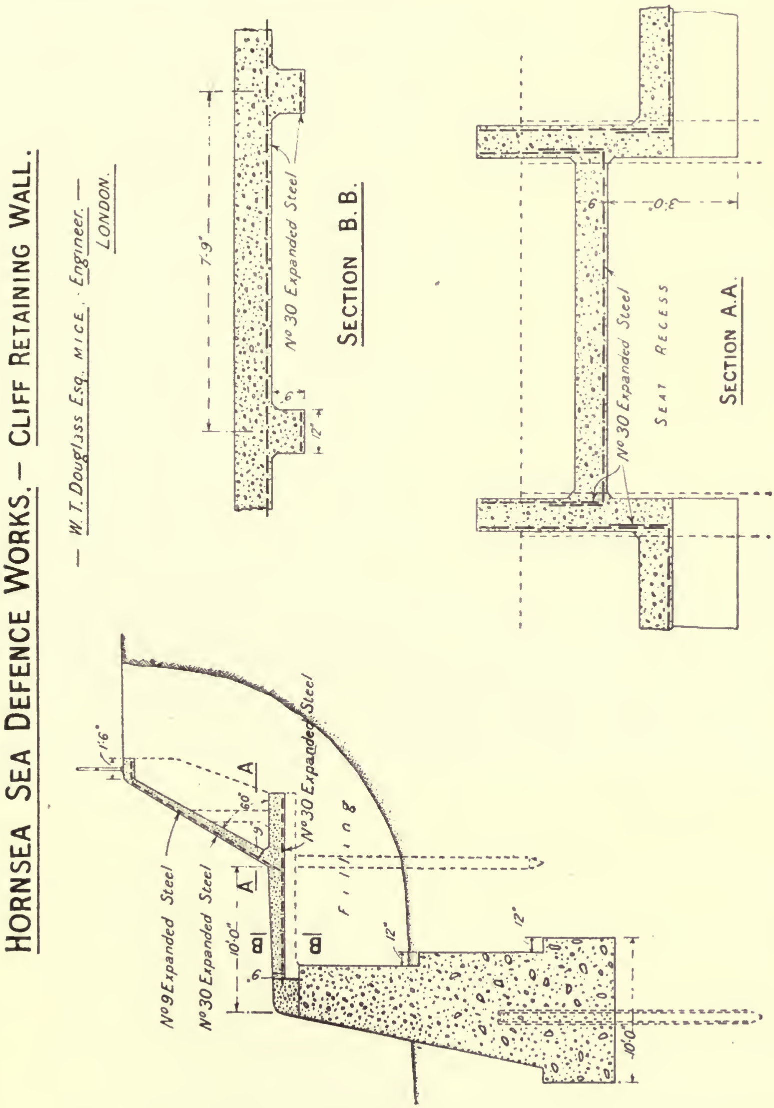


Fig. 272.

The superstructure is supported by concrete columns, and main girders which are 2 ft. in depth by 1 ft. wide.

The stairs, approaches, towers—in fact, the whole structure is built in reinforced concrete, and is illustrated in Fig. 271.

Sea-wall Construction.—One illustration only will be given of the use of this material in the construction of sea-walls. The new sea-wall at Hornsea, Yorkshire, designed by Mr. W. T. Douglas, M.Inst.C.E., of Westminster, is a very good example. This is illustrated in Figs. 272, 273.

The reinforcement used was expanded metal, 3 in. mesh.

Fig. 273.

The cliff wall averages 13 ft. 6 in. high, the foundation platform is 14 in. thick at the back tapering to 9 in. at the front. The counterforts are at 7 ft. 9 in. centres, and are 9 in. thick; the face wall is 9 in. thick at the toe and tapers to 4½ in. thick at top, where it returns to form a coping 1 ft. 6 in. in width, and 6 in. in thickness.

Reinforced concrete has been used for a score of other useful engineering purposes, but space does not permit of the authors describing or illustrating these.

CHAPTER X.

REINFORCED CONCRETE IN BUILDING CONSTRUCTION.

NOT only is this material eminently suitable for use in the construction of engineering works, but it may be used with equal advantage in building construction, and has been used very extensively for this purpose.

The authors propose in this chapter to give typical examples of its use in all classes of buildings, and for a variety of purposes, including piles, foundations, columns, floors, roofs, beams, stairs, balconies, walls, partitions, retaining walls, etc.

PILES.

Coignet System.—Reinforced concrete piles are a valuable substitute for the ordinary timber piles which are subject to deterioration. One of the first to recognize this advantage was Mr. Edmond Coignet, who made two reinforced piles in 1894; these were driven in connection with the foundations of the Generating Station of the Champs Elysées, Levallois Perret, Paris.

Their length was about 16 ft., and diameter 10 in., they were examined some time after having been driven, and were found to be in excellent condition. Many piles have since been driven on this system. The piles are concreted in a horizontal mould instead of a vertical one as in the case of some systems. Coignet piles are generally of a circular section, varying between 10 and 16 in. in diameter.

Piling at Tobacco Warehouses, Bristol.—The arrangement of the reinforcement is shown in Figs. 274, 275, which represent piling used in the foundations for a tobacco warehouse at Bristol, where 650 piles were driven.

It was necessary that the piles should be of considerable length in order to reach a stratum of gravel situated at a depth of about 45 ft. below the surface.

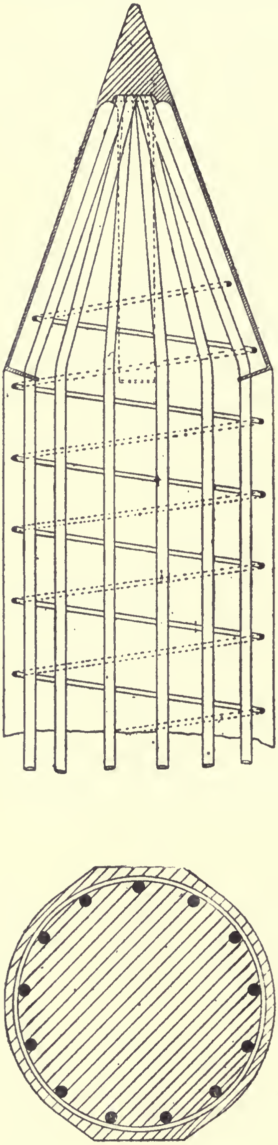


Fig. 274.

It should perhaps be mentioned that circular piles are easier to drive than square ones, and that as they have no sharp edges they are not likely to be broken by coming into contact with boulders.

The area of the foundations which were required to carry the warehouse the authors are referring to, was approximately 215 ft. by 102 ft.; the piles were calculated for a safe load of 56 tons each; some of them were tested to 90 tons before being driven, without showing any sign of failure.

The piles were 14 to 15 in. in diameter, with two flat surfaces of about 5 in. wide for guiding purposes in driving. The reinforcement consisted of a number of longitudinal bars of small diameter bound by a spiral wire, and the spiral was bound to the bars with annealed wire.

Each pile was shod with a cast-iron shoe which was first placed inside the mould.

The concrete was machine mixed, the aggregate consisting of sand and granite chippings.

The piles were moulded on a flat sill, to which were bolted the two semi-circular detachable sides. The concreting operation was carried out horizontally, the framework being suspended inside the mould by the upper bolts.

The sides were removed as soon as the concrete had begun

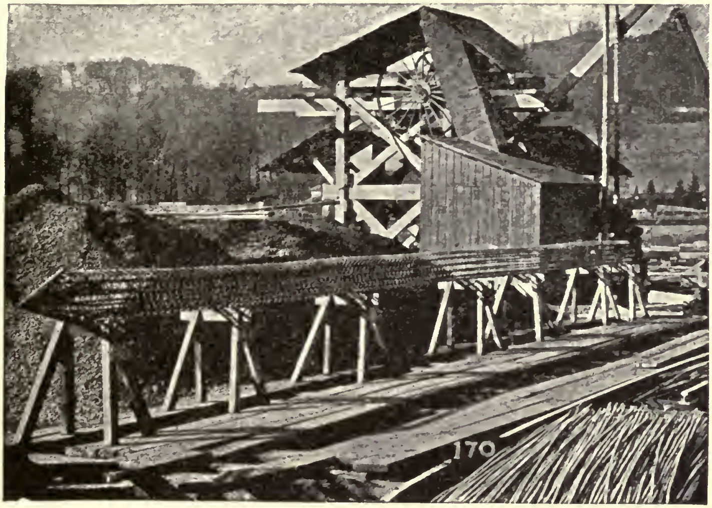


Fig. 275.

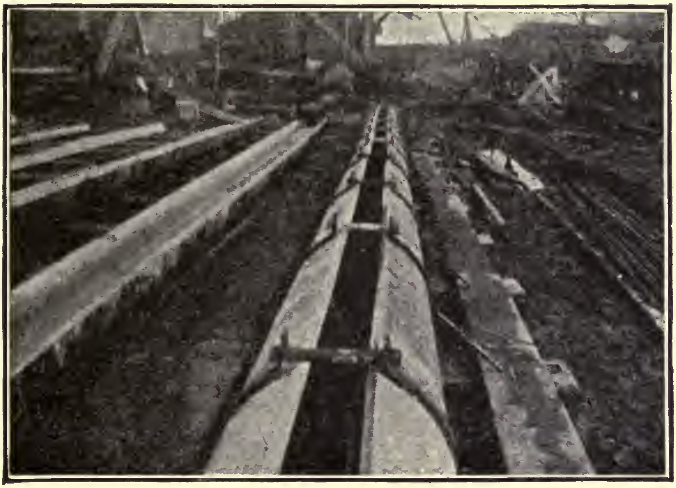


Fig. 276.

to set, the pile being allowed to season for a fortnight before removing it from the sill.

Each pile weighed about 5 tons.

Six weeks were required for the proper seasoning of the piles before the driving operation.

Method of Driving.—The piles were driven in the following manner:—

They were pitched by firmly securing them at about one-third of their length and gradually lifting them to their proper position; the head of the pile was then fitted with a wooden dolly in order to prevent the ram from injuring the concrete. The weight of the ram was 2 tons, with a drop of about 4 ft. The enormous strength of these piles is shown by the fact that some of them received over 2000 blows without injury. Two driving machines was used, and the work, when nearing completion, was also carried on at night; in this manner it was possible to drive about 12 piles per twenty-four hours.

The piles were driven in groups of six, and a reinforced concrete cap was provided in order to evenly spread the load of the pillars supporting the floors. The weight on each cap is about 300 tons, and the caps are connected by reinforced concrete beams.

The advantages of using reinforced concrete piles are that a pile of this description will carry about twice as heavy a load as that of a timber pile; it can be driven to a much greater depth, and is practically indestructible. A timber pile of similar section may be less expensive, but considering the advantages of the reinforced concrete pile it is in the end most economical.

The foundations for the Dundee Generating Station, and of many other important buildings, are on the Coignet system. The piling at Bristol above referred to was executed by Messrs. W. Cowlin & Son, contractors, Bristol, licensees of the Coignet system, the work, executed for the Bristol Corporation, being designed by, and carried out under the supervision of, the Docks Engineer, Mr. W. W. Squire, M.Inst.C.E.

Paragon System.—The British Reinforced Concrete Engineering Co., of Manchester, make a similar pile, except that it is octagonal in section, and sometimes square. These piles are made from 30 to 45 ft. in length, and are each capable of carrying a safe load of 50 tons. Fig. 277 represents one of such piles, the special feature of which is the helical reinforcement, which is so arranged that a pile may be built up in sections, the helicals being sectionized. At certain distances are fixed frame hoops for keeping the longitudinal bars in place.

Considère System.—The method of increasing the strength of concrete in compression whether in piles or columns by the introduction of helical reinforcement was really invented by M. Considère, and after repeated independent tests has been officially recognized in the Regulations of France, Germany, and Austria; detailed accounts of these tests on spiralled concrete may be found in the Report of the French Commission on the subject, as well as in the writings of Prof. Bach, Von Thullie, Mörsch, and others.

It has been conclusively proved that spiral reinforcement, in addition to giving a much greater safeguard against the effects of bad workmanship, is, weight for weight, 2·1

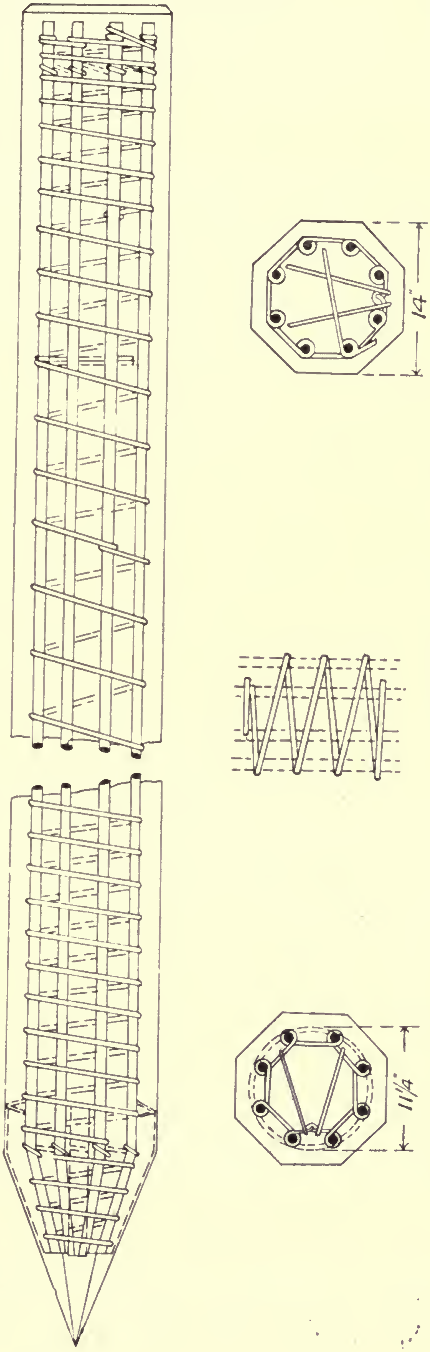


Fig. 277.

to 2·4 times more efficient than longitudinal rods and links, or ties of the usual type. A special feature of the Considère piles

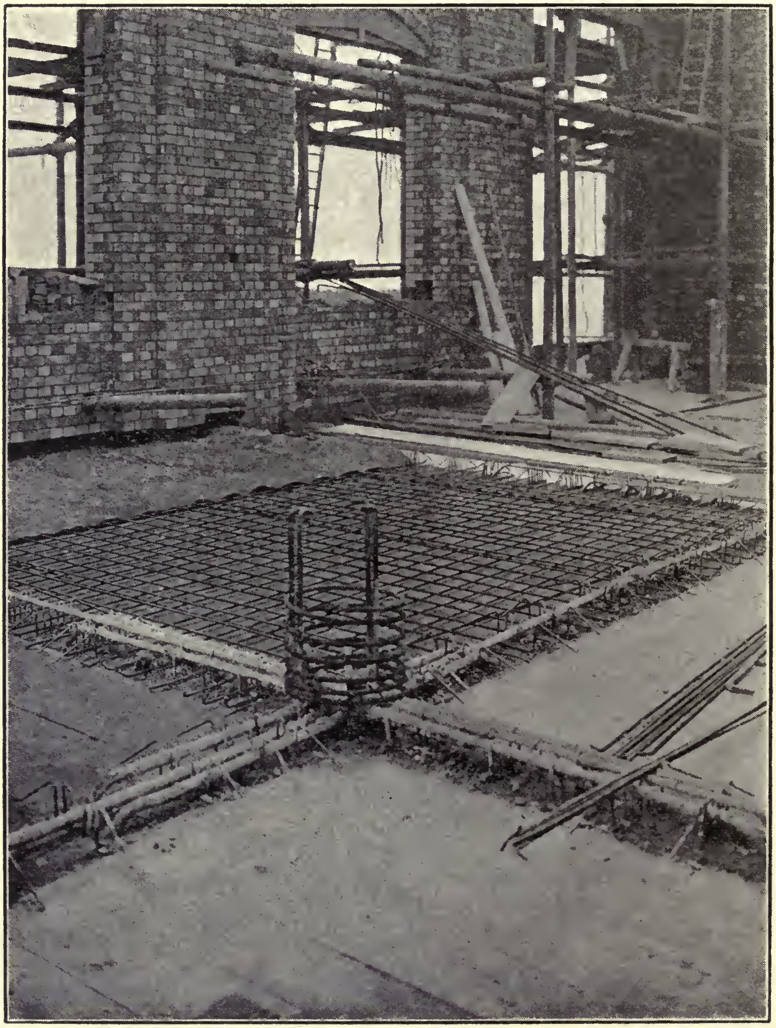


Fig. 278.

is that in driving them it is not necessary to protect the pile with a sawdust-filled cap over the head, and a timber dolly.

M. Considère points out that driving them in the manner adopted

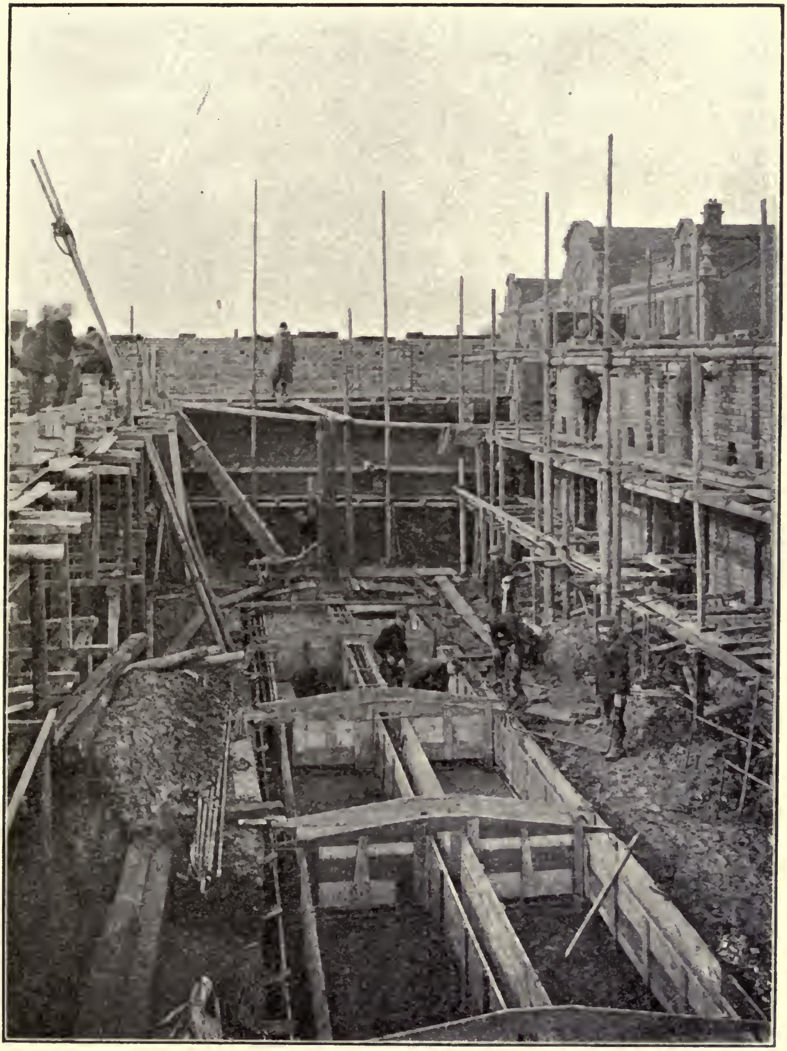


Fig. 279.

by him is more economical and efficient. The system is illustrated in Figs. 278 to 281.

Hennebique Piles.—These have been used chiefly in the construction of wharves and jetties, and have been referred to

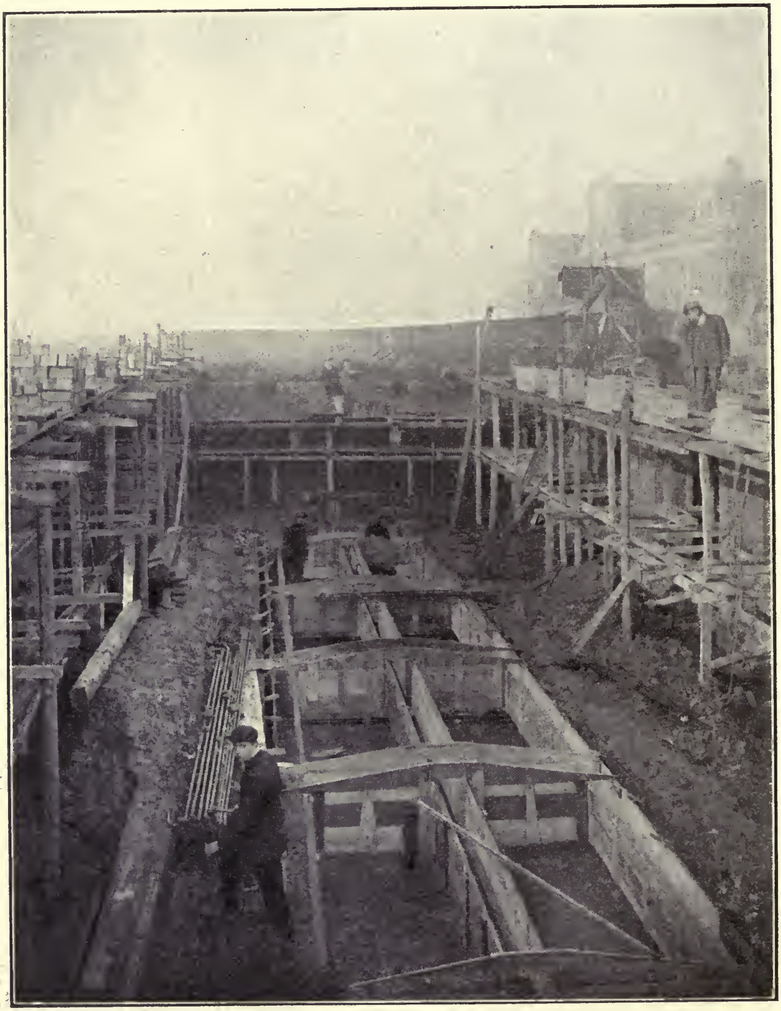


FIG. 280.

in a previous chapter. They are square in section, the angles being splayed and the top of the pile is of a round section to admit of the application of a driving-cap. The reinforcement consists of a longitudinal rod at each corner, wired together at intervals.

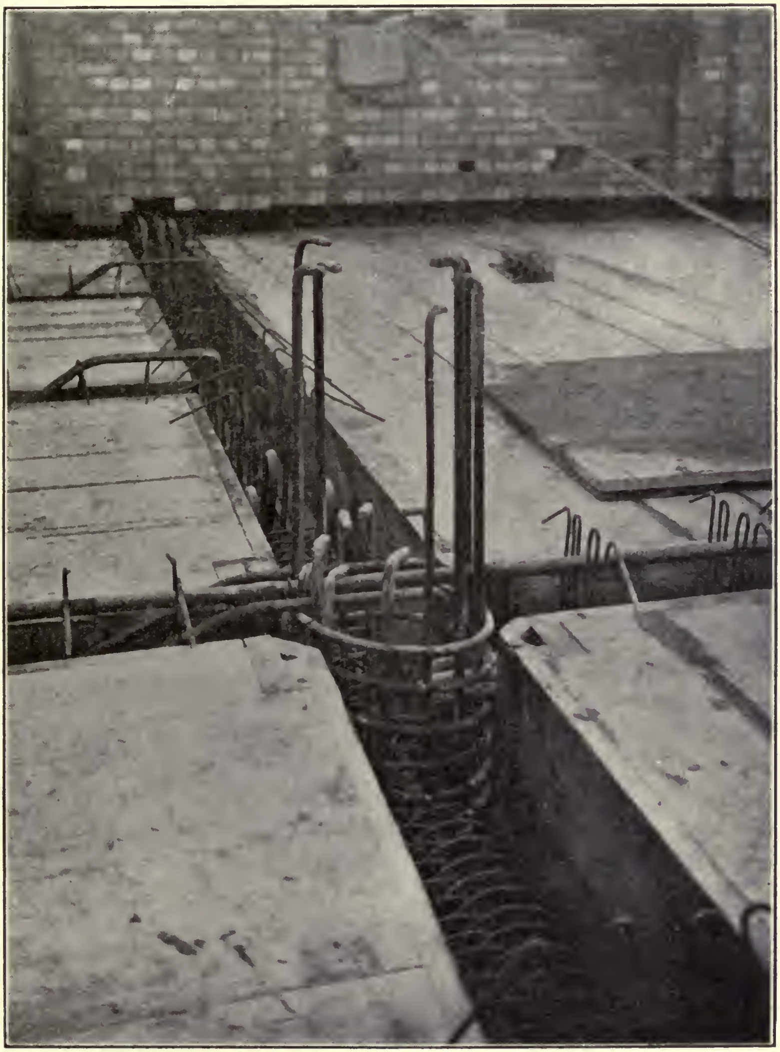


FIG. 281.

Simplex and Raymond Piles.—These are the two piles used chiefly in America. The first is built in place, and is reinforced by a cylinder of expanded metal, the latter is reinforced by a round rod about $1\frac{1}{2}$ in. diameter which passes along its axis, and three $\frac{3}{4}$ in. bars placed around the circumference. The "Simplex" pile is the patent of the Simplex Concrete Piling Co., of Philadelphia, Pa., the "Raymond" pile is the patent of the Raymond Concrete Pile Co., of Chicago, Ill.

Columns, Beams, and Floors.—These three portions of constructional work are taken together, and the illustrations which are given show the connection which is made between the column, beam, and floor.

The various systems previously described of reinforcement for piles, are also suitable for the construction of columns, and are largely used for that purpose.

Columns, Beams, and Floors on the Coignet System.—A good example of the construction of columns, beams, and floors on this System may be found in the Tobacco Warehouse at Bristol previously referred to. Fig. 282 represents a view of the ground floor of this building. The doors are all fire-proof, and the walls of lifts are built in reinforced concrete. The stairs and landings are also built in this material, and the columns, beams, and floors.

The floors are calculated for a superimposed load of $1\frac{1}{2}$ cwts. per square foot.

Another excellent example of the use of this System in building construction may be seen in the new Money Order Department for H.M. General Post Office, which is fully described in "Concrete and Constructional Engineering," 1910.

The building was designed by Sir Hy. Tanner; the Contractors being Messrs. W. King & Son.

Leslie's System.—The system of reinforcement for columns invented by Messrs. Leslie & Company, Ltd., of London is clearly seen in Fig. 283.

Leslie's system of beam and floor construction is also shown in Fig. 283.

This system includes reinforcement with vertical hangers rigidly attached by links or rings to the main reinforcement. The whole of the steelwork is framed up completely as a unit, so as to be capable of inspection before concreting, and so that it may not be displaced when the concrete is put in place,

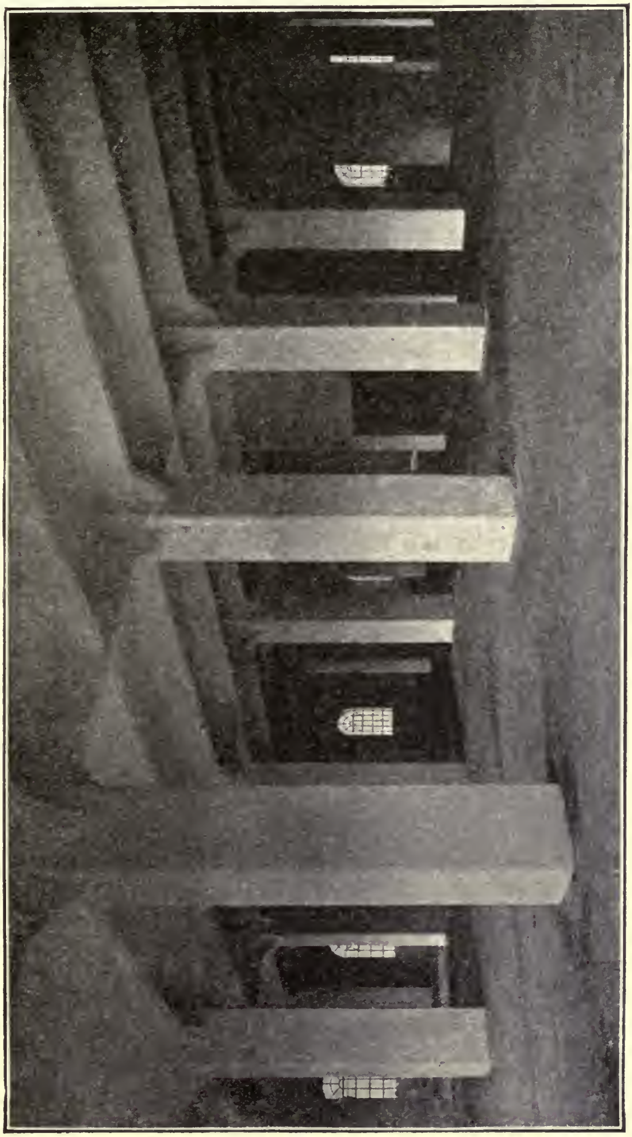


Fig. 282.

There is complete truss action in beams constructed on this system.

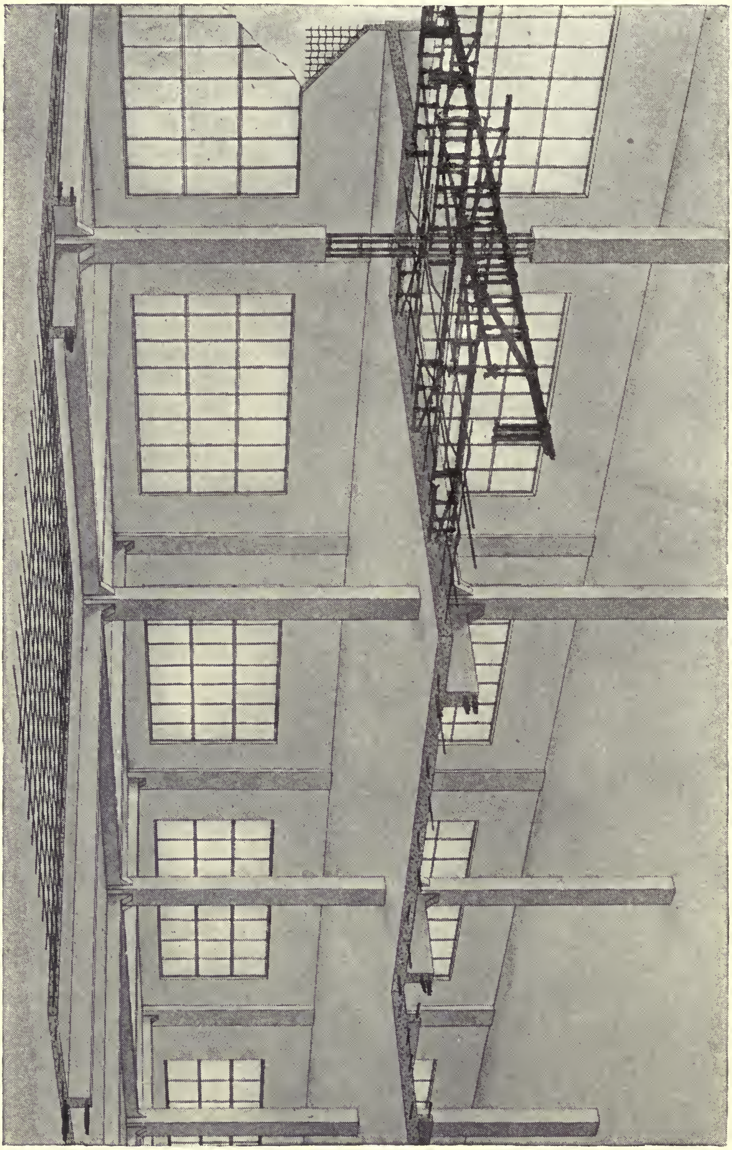


Fig. 283.

Figs. 283, 284 represent the interior of a portion of a new factory at Messrs. Bryant & May's, Ltd., Bow, London, E., constructed on this system, the architects being Messrs. Holman & Goodrham.

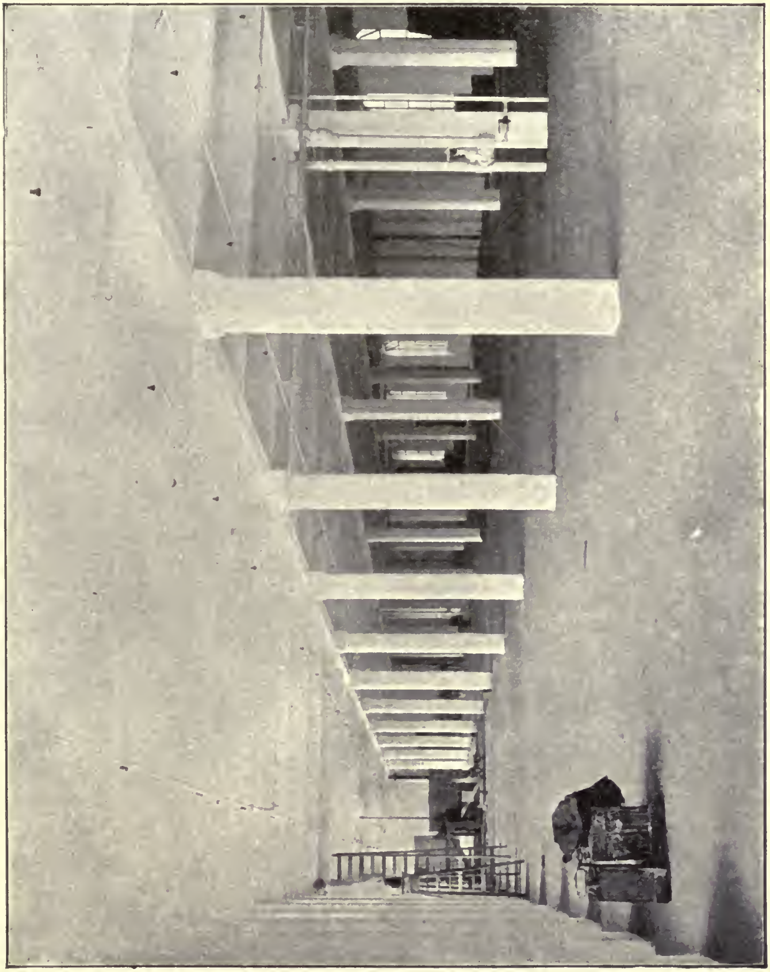


Fig. 284.

The Chain Concrete Syndicate's System.—This has already been described and illustrated in the chapter dealing with Municipal Engineering. The authors would, however, here insert a view of the inside of a factory at Cleckheaton, where the columns, beams, and floors have been constructed on this system, the architects being Messrs. Howarth & Howarth of Cleckheaton.

FIG. 285.

Mr. E. P. Wells' System.—Stuart's Granolithic Co., Ltd., have carried out a great deal of similar work on Mr. Wells' system; some of the most interesting being the factory erected at Portobello, N.B., for Messrs. Schulze & Co., manufacturers of chocolate confectionery, and a factory at Hayes, Middlesex, for the London Orchestrelle Co., the Architect of which was Mr. Walter Cave.

The Royal School of Electricity at Chatham for the War Office, and a large factory near Edinburgh, are also examples of work on this system.

Fig. 286 illustrates the Portobello building, and Fig. 287 the factory at Hayes.

Expanded Metal Co.'s System of Floor Construction.—Various methods of floor and beam construction are suggested by this firm, the reinforcement being Expanded Metal, with the addition of rods in the case of beams.

Fig. 288 represents a floor slab carried by an offset in the wall.

Fig. 289 shows a slab carried by a corbel.

Fig. 290 shows one carried by a chase.

Fig. 291 illustrates various methods of inserting the reinforcement in floor and beam construction.

Figs. 292 and 292*a* represent the use of expanded metal in the construction of sea defences.

British Reinforced Concrete Engineering Co.'s System.—The accompanying drawings illustrate the method of placing the reinforcement in columns, beams, and floors on this system (see Figs. 244, 245, and 246).

Fig. 286.

Kahn System.—The Trussed Concrete-Steel Co.'s system known as the Kahn System is well illustrated in the important work now being executed at the Wesleyan Methodist Hall, Westminster, S.W.

Drawing No. 297 illustrates one of the main arched beams. The placing of the reinforcement is clearly shown.

Fig. 287.

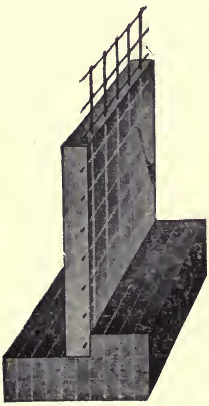


Fig. 288.

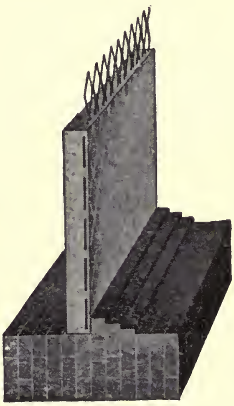


Fig. 289.

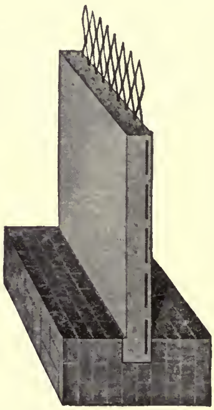


Fig. 290.

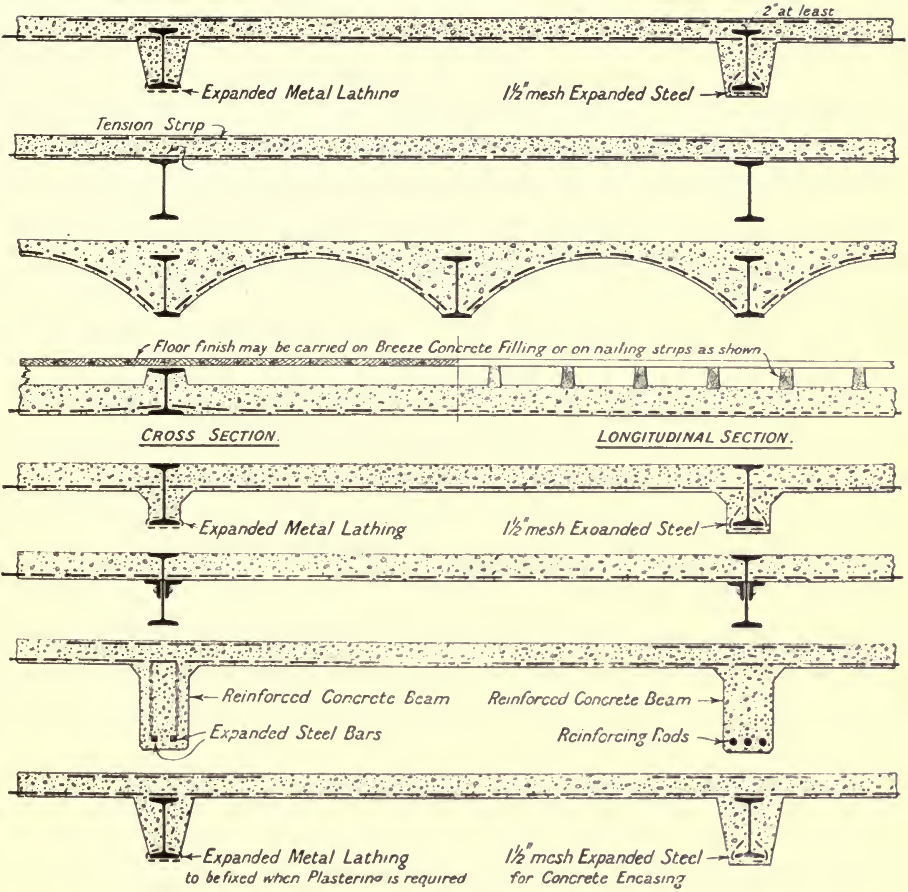


Fig. 291.

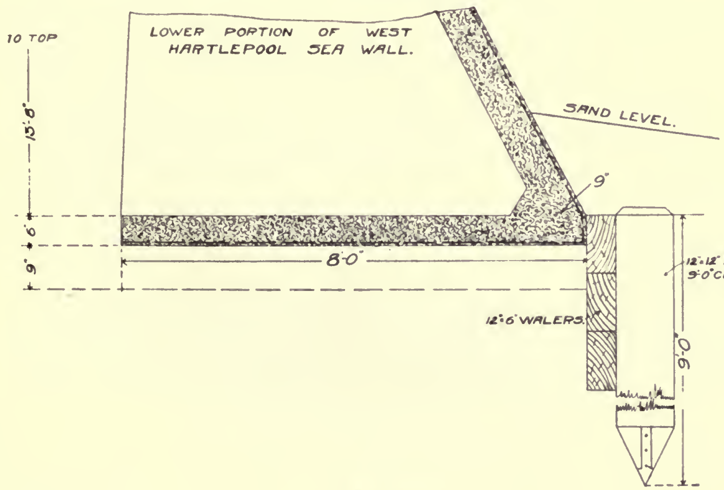


Fig. 292.

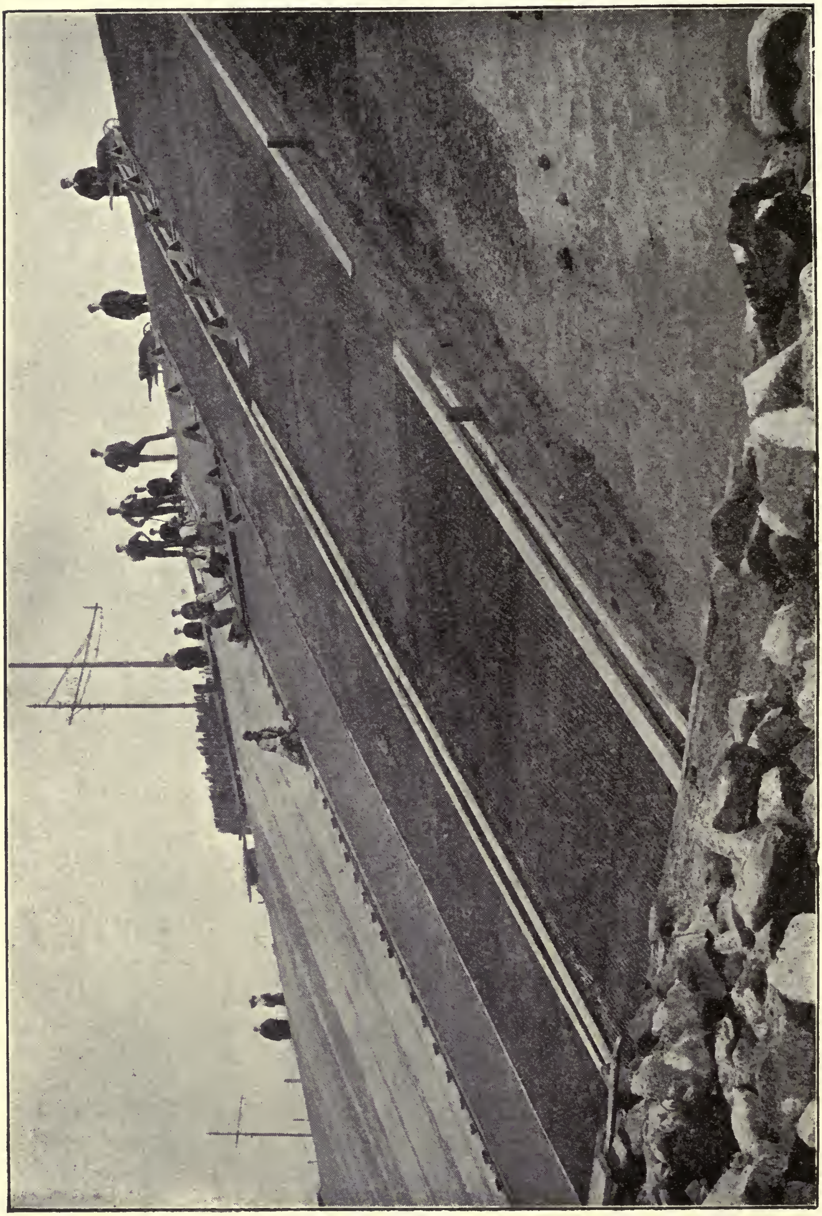


Fig. 292a

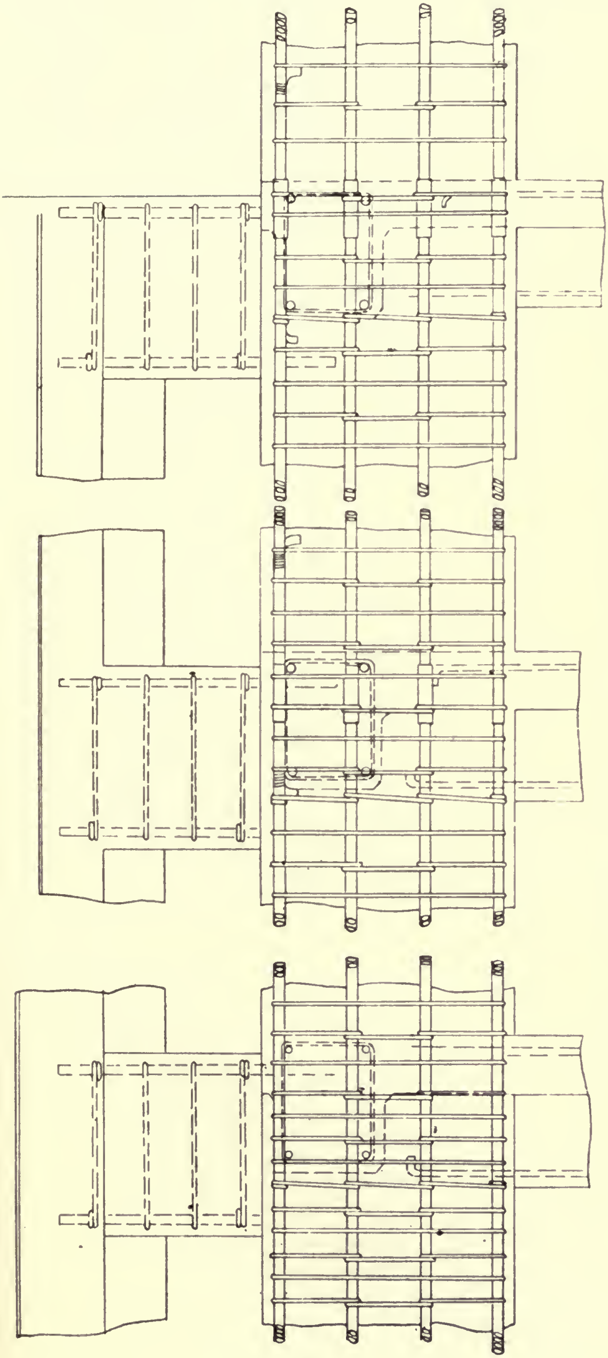


Fig. 293.

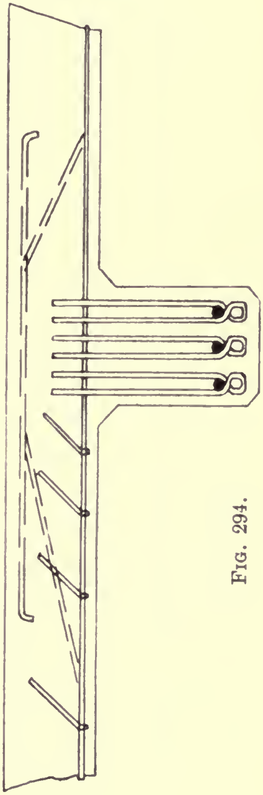


Fig. 294.

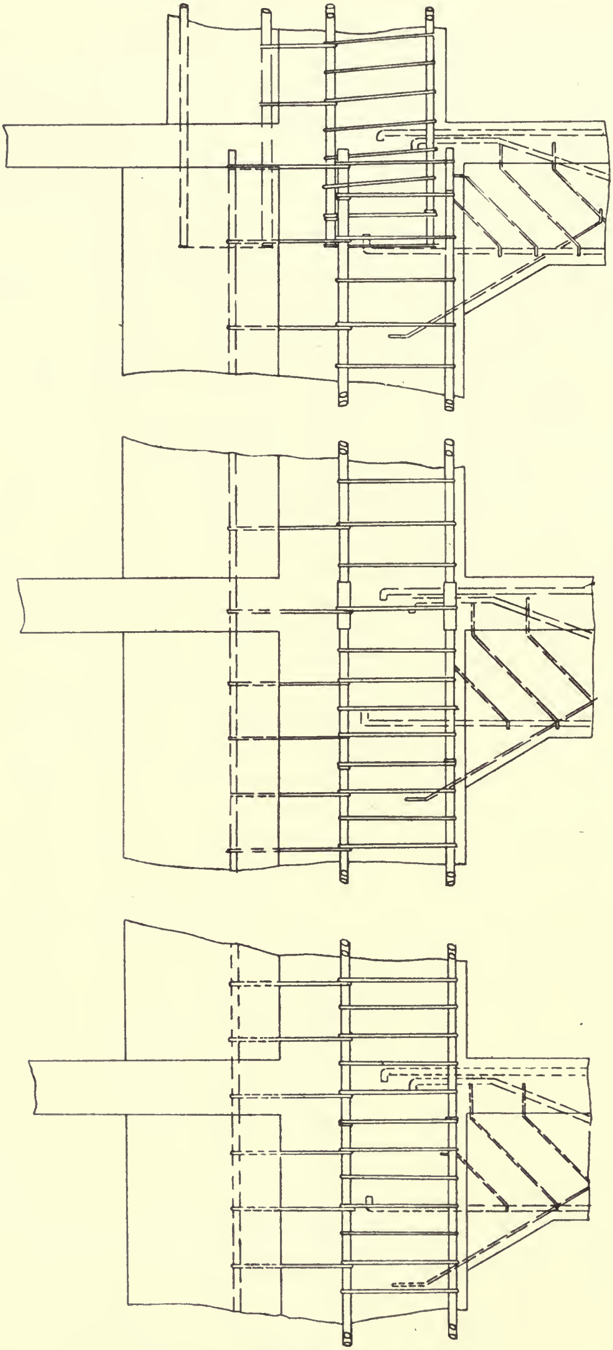


Fig. 295.

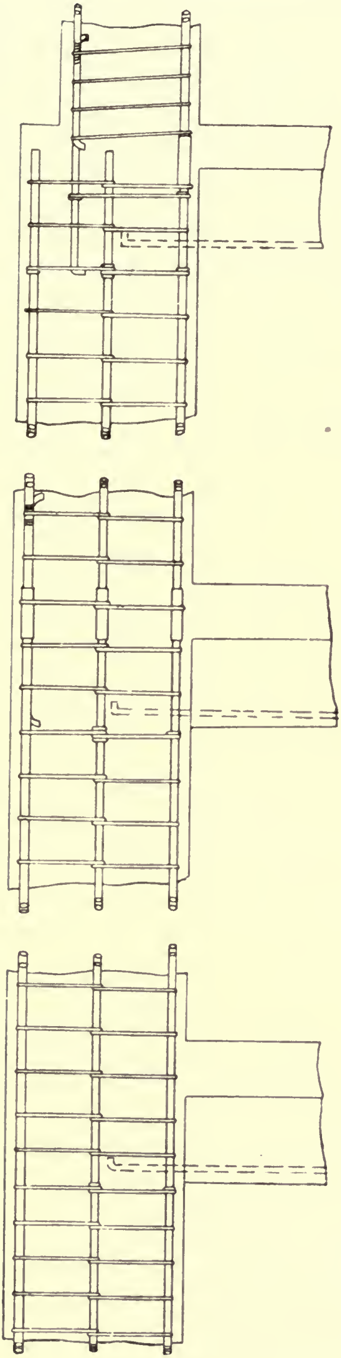


FIG. 265.

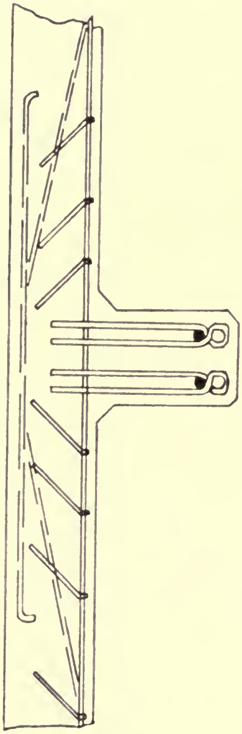


FIG. 296.

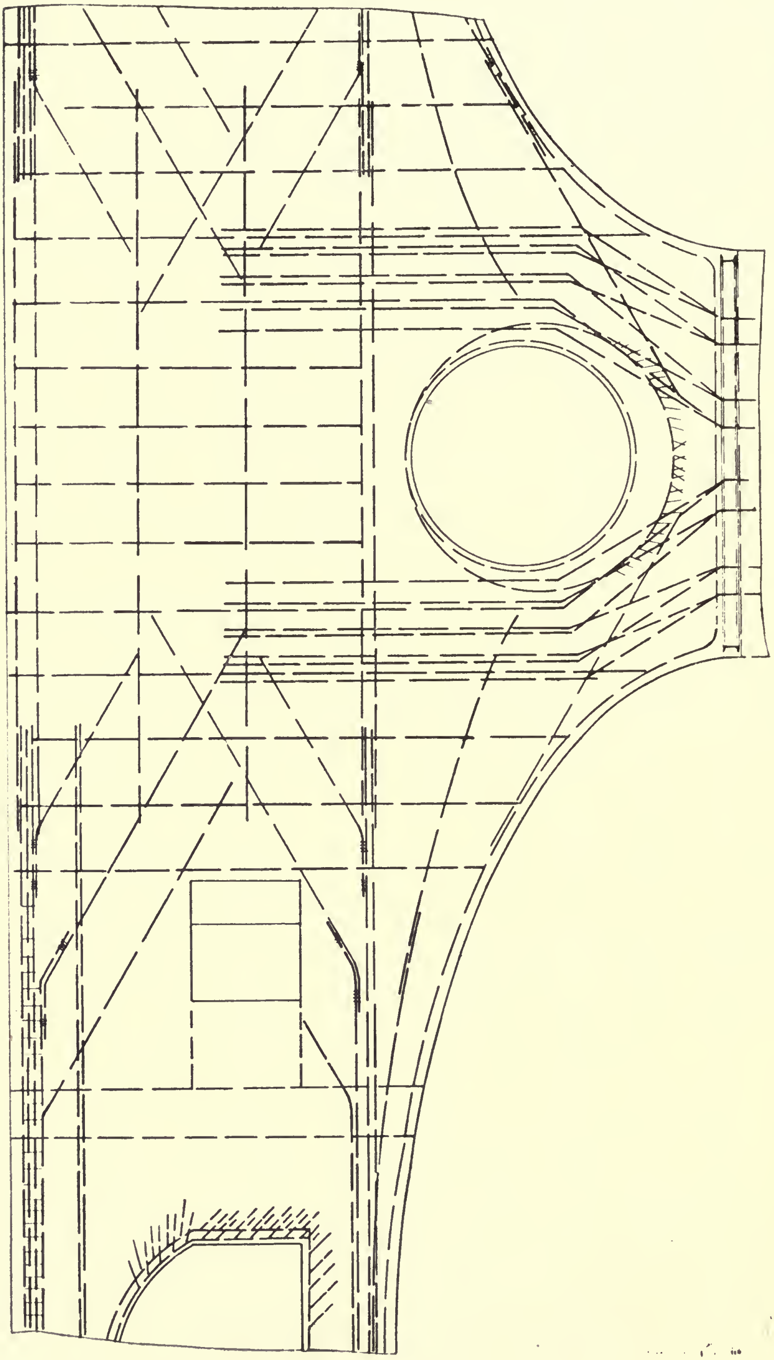


Fig. 297.

Drawing No. 298 shows the beams carrying floor over Tea Room in the same building.

The Architects for this important work were Messrs. Lanchester and Rickards, London.

Drawings Nos. 299 and 300 show the reinforced concrete columns, beams, and floors on the Kahn System supporting and forming the gallery of the Stoke-upon-Trent Town Hall, which has recently been extended.

Hennebique System.—The general arrangement of the reinforcement in this system is shown in the diagram, Fig. 302*a*, which represents a reinforced column supporting a beam, details of a column, etc.

The special feature in this system is the patent stirrups used.

A great many excellent examples of work carried out on this system might be described; illustrations of three, however, only will be given.

Fig. 301 illustrates the R.C. arched beams and flooring at the building of the Yorkshire Mutual Garage Co., Ltd., Leeds. The beams have a span of 41 ft.

Fig. 302 shows the columns and underside of ground floor in the new Almond Block, at Messrs. J. Rowntree & Co.'s Works, York.

Fig. 303 represents the columns and underside of floor in the New Warehouse of the Carrongrove Paper Co., Ltd., at Denny, N.B. The beams have a span of 34 ft.

Patent Indented Bar Co.'s System.—A good illustration of work carried out on this system may be seen in the case of the erection of Rawson's Factory, Leicester, where the footings, columns, beams, floors, and roof were built in reinforced concrete.

Mr. J. H. Simpson was the Architect.

Foundations.—The authors have already dealt with piled foundations, they will now briefly describe the use of this material in the ordinary foundations of buildings, and they will give several examples of the use of reinforced concrete for purposes of this kind.

The material is employed in a variety of ways in foundation

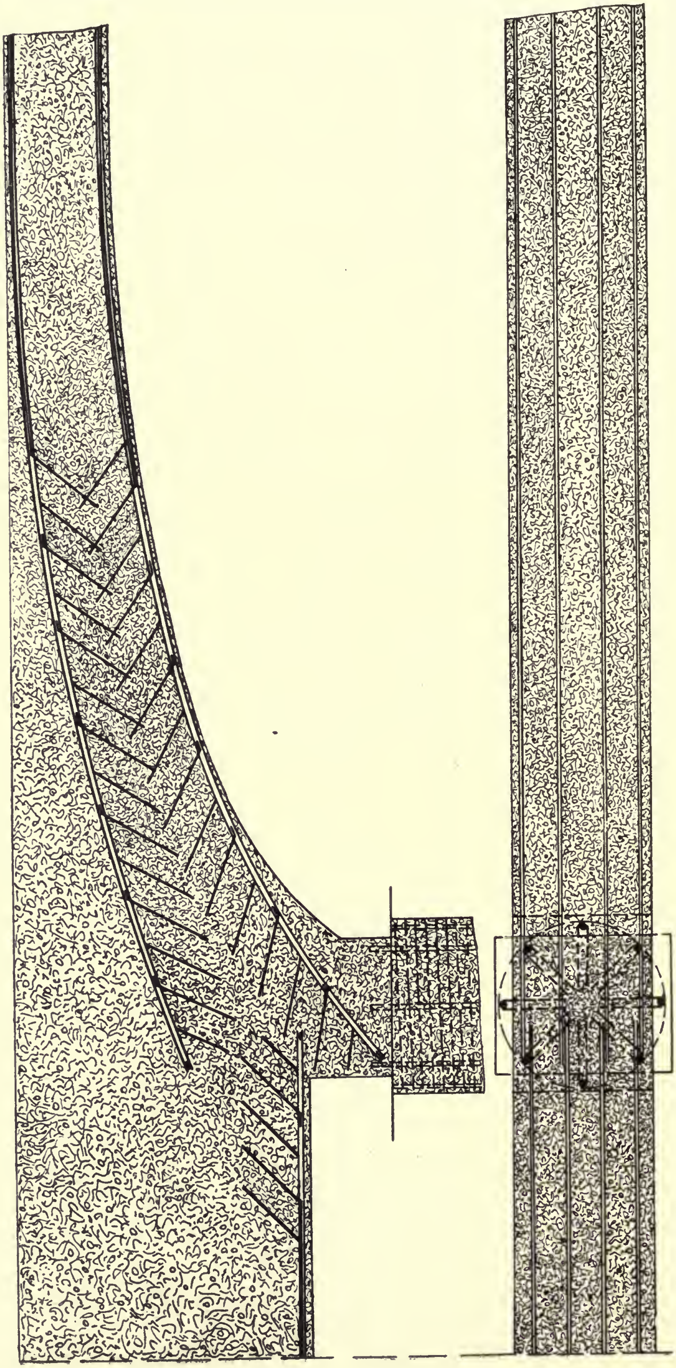


Fig. 298.

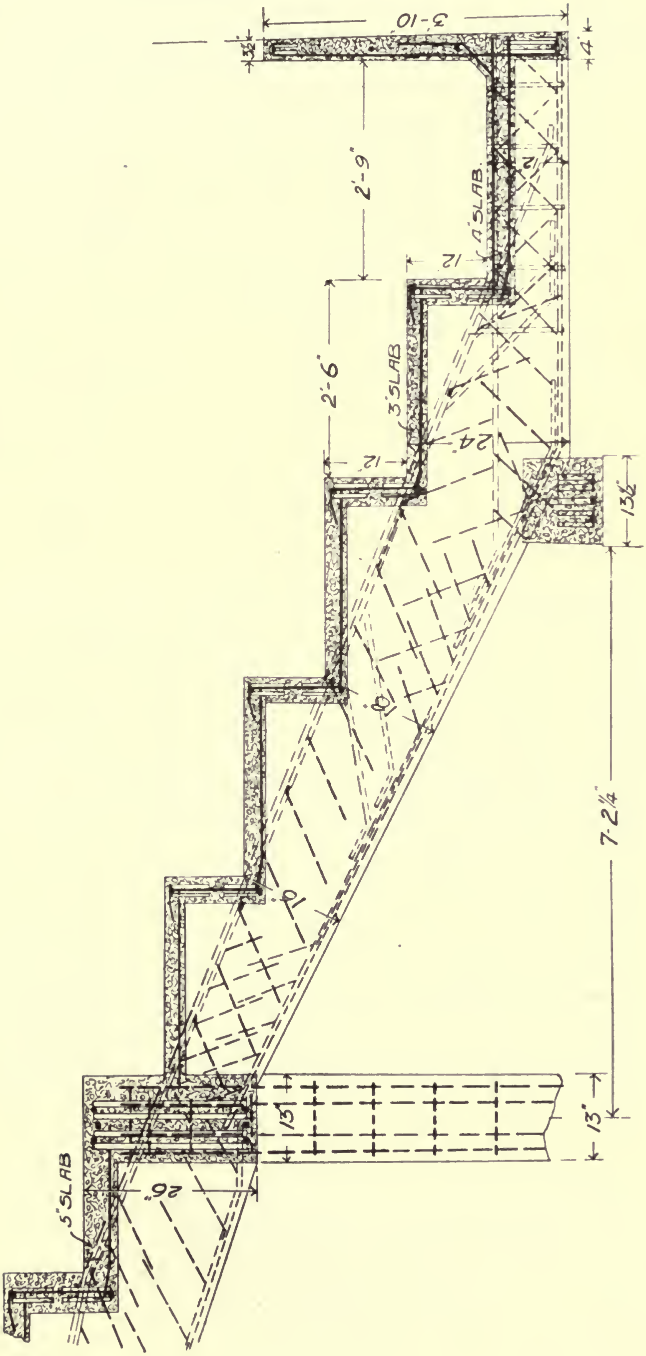


FIG. 299.

work. Where piling has to be resorted to, the tops of the piles are usually run into a reinforced beam, which connects the whole of them, and upon which the structure is erected. Sometimes a beam of this description connects the tops of timber

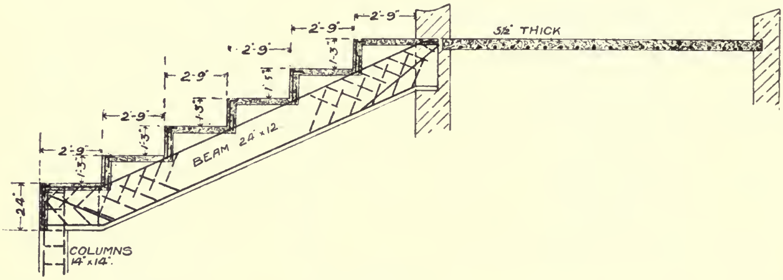


FIG. 300.

FIG. 301.

piles, but such an arrangement is never so satisfactory, for in time the heads of the piles rot, and the beam is therefore considerably reduced in strength.

It is also occasionally necessary, on a doubtful foundation, to construct a reinforced concrete raft or platform. One of the authors found it necessary to construct such a raft in connection with the foundations of a tall chimney at Bridlington. The concrete in this case consisted of a 5 to 1 mixture (5 of gravel to 1 of cement), and the reinforcement of 9 in. by 6 in. rolled steel joists, bolted together by means of cross joists 9 in. by 6 in. being secured to them. The chimney, which is situated at the Bridlington Electricity Works, is about 150 ft. in height above foundations, and is 9 ft. internal diameter, and built in brick.

Foundations similar to this have been frequently put in in connection with the construction of tall buildings in Amercia, and the result has been found to be very satisfactory.

The foundations of Sprecles Building, San Francisco (a nineteen storey building) is of this type.

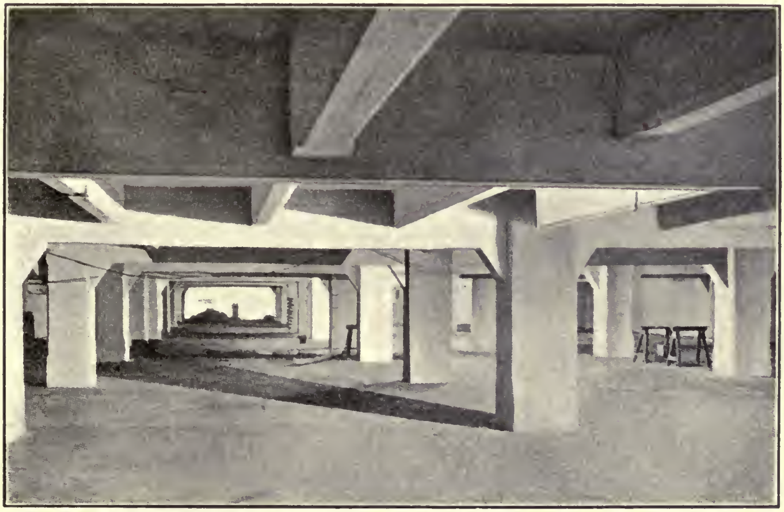


FIG. 302.

A good example of a reinforced concrete raft to form the foundations of an important building in England, is seen in connection with the foundations of the Stoke-upon-Trent Town Hall Extensions.

This is carried out on the Kahn System.

The Patent Indented Steel Bar Co.'s method of reinforcing foundations for columns or walls is seen in Fig. 304.

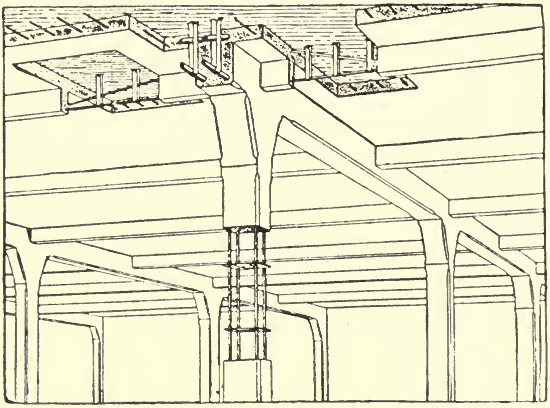


Fig. 302*a*.

Fig. 303.

The Expanded Metal Co. recommends the use of expanded metal in foundations on any of the lines indicated in the diagrams on pages 284, 285.

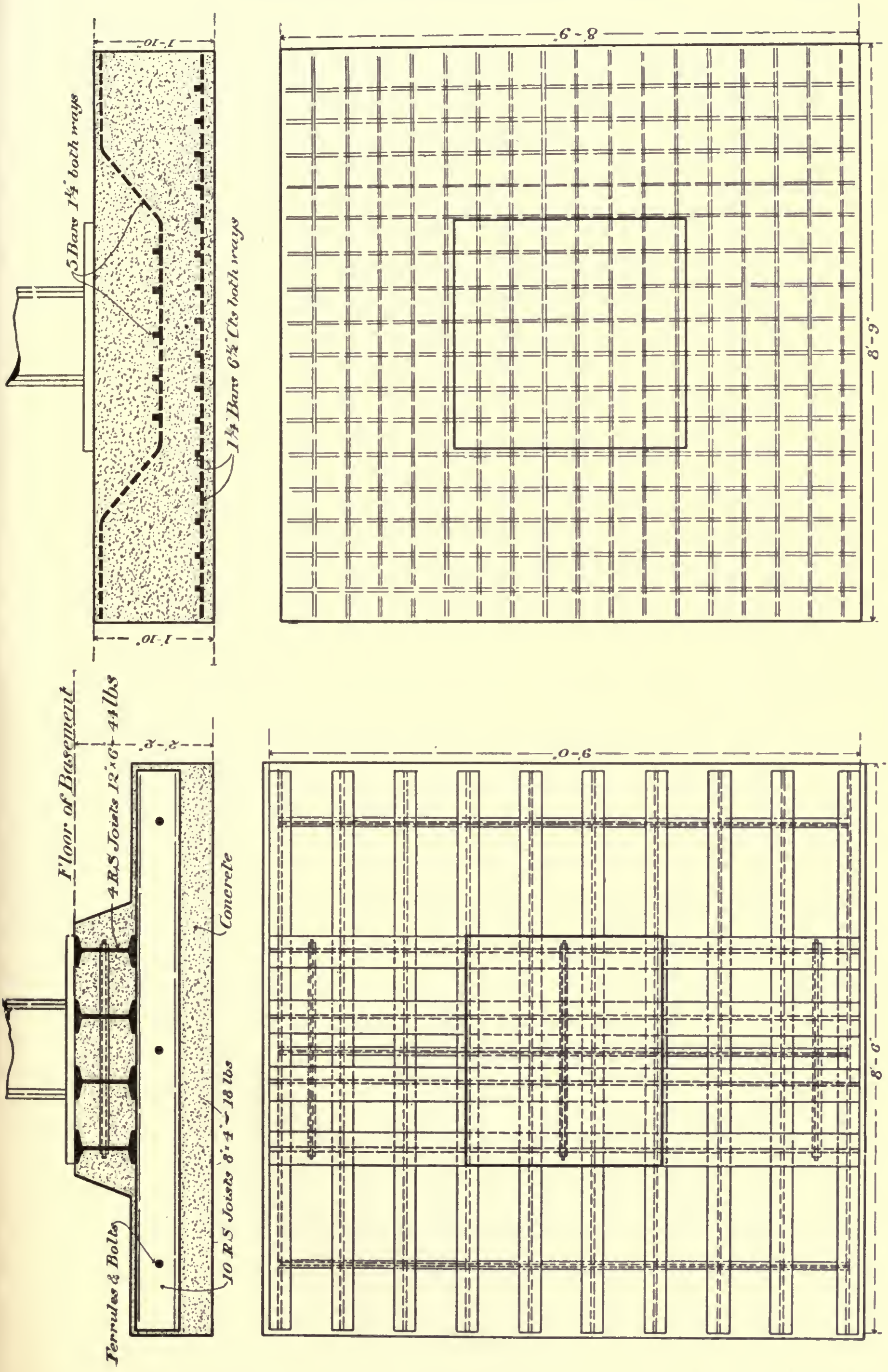

1'-10"
5 Bars 1¼" both ways
1¼" Bars 6¼" Cts both ways
8'-9"
Floor of Basement
4 R.S Joists 12"×6" – 44 lbs
Concrete
10 R.S Joists 8"×4" – 18 lbs
Ferrules & Bolts
2'-8"
9'-0"
8'-6"

Fig. 304.

EXPANDED STEEL-CONCRETE WALL FOOTINGS

SOME EXAMPLES.

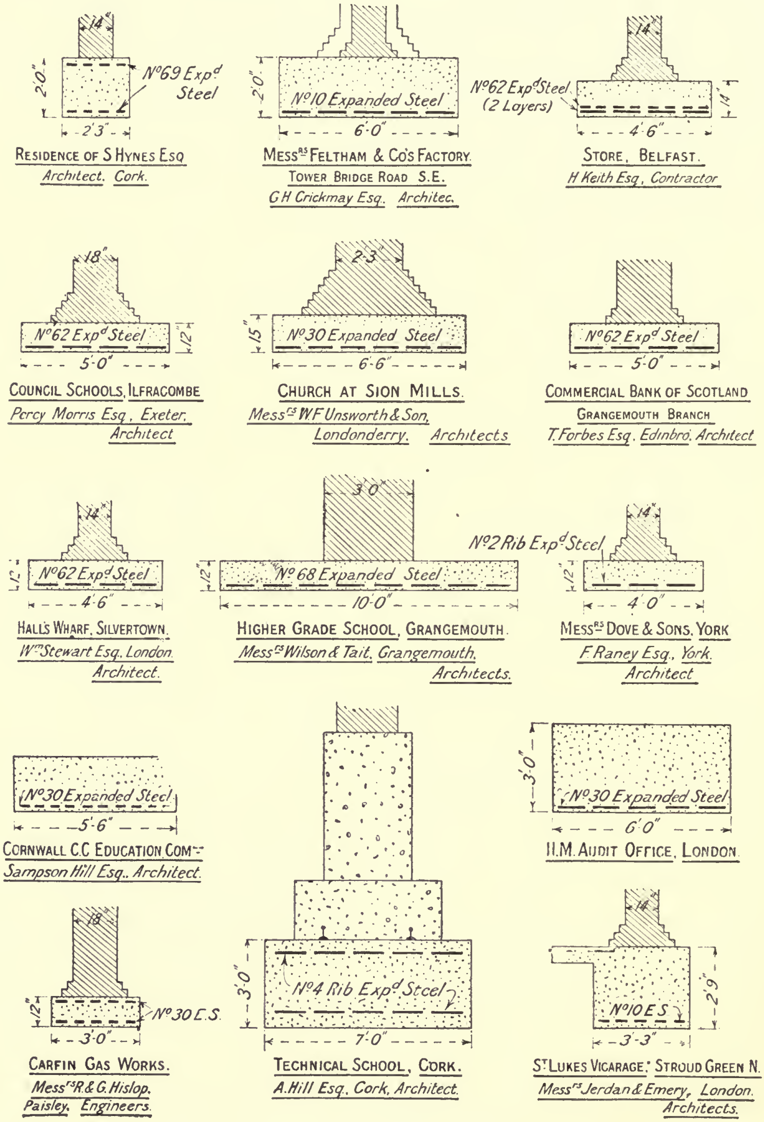


FIG. 305.

This material has been used very largely in foundation work.

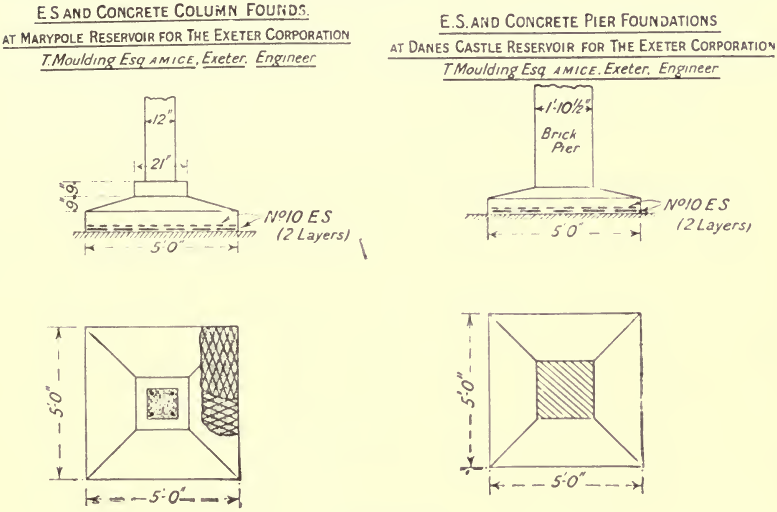


FIG. 306.

Walls and Partitions.—A very great saving is effected by the use of reinforced concrete in the construction of walls and foundations; a reduction in cost occurs, and also a saving of valuable space. The latter has been set out by one of the authors in a paper read before the Royal Society of Arts,[1] in which he says:—

"The regulations (1906) of the City of Buffalo specify that the thickness of the reinforced concrete walls of a building shall be as follows: Where there is a basement: If one storey, 8 in.; if two storeys, 10 in.; if three storeys, 12 in. Where there is no basement: If one storey, 6 in.; if two storeys, 6 and 6 in.; if three storeys, 8, 6 and 6 in.

What an improvement is, therefore, effected in respect to additional space obtained by using reinforced concrete. Assuming, for example, that a warehouse is 70 ft. in length, 33 ft. in width, and has three storeys, each of which is 12 ft. in height.

[1] "Reinforced Concrete in Engineering and Architectural Construction in America," by E. R. Matthews, read before Roy. Soc. of Arts, March, 1908.

It has no basement. The thickness of the walls, if of reinforced concrete, would be, first storey 8 in., second 6 in., third 6 in.; but if the walls were built of brick, then, taking say the city of Birmingham regulations as being a fair example of our British regulations, these being up-to-date, the thickness of the longitudinal walls of the warehouse under consideration would be as follows: First storey $22\frac{1}{2}$ in. thick, second storey 18 in.; third storey $13\frac{1}{2}$ in.; the thickness of end walls would be: first storey 18 in. thick, second storey 18 in., third storey $13\frac{1}{2}$ in. By using reinforced concrete, under the American building regulations, there would be an increase of floor area on the ground floor of 227·48 sup. ft. made up as follows: Floor area with reinforced concrete walls, 71 ft. 8 in. by 35 ft. 5 in. = 2537·48. Floor area with brick walls, 70 ft. by 33 ft. = 2310·00. Increase of floor area on ground floor, 227·48 sup. ft. An increase of floor area would occur on the first floor of 211·5 sup. ft., as follows: Floor area with reinforced concrete walls, 72 ft. by 35 ft. 9 in. = 2574·00. Floor area with brick walls, 70 ft. by 33 ft. 9 in. = 2362·50. Increase of floor area on first floor, 211·50 sup. ft. On the second floor there would be a saving in floor area of 133·13 sup. ft. Foor area with reinforced concrete walls 72 ft. by 35 ft. 9 in. = 2574·00. Floor area with brick walls 70 ft. 9 in. by 34 ft. 6 in. = 2440·87, a difference of 133·13. So that the total floor area saved by building the walls of reinforced concrete instead of brick would be 572 sup. ft."

Roof Construction.—Most of the systems of reinforced concrete floor construction already referred to are also suitable for roof construction, the bars or other reinforcement being usually of lighter section owing to the lesser weight that will come upon a roof than that which a floor would have to carry. It sometimes happens, of course, that a flat roof will be required to carry quite as heavy a load as a floor, in which case it will be designed as a floor.

A mansard and flat reinforced concrete roof are sometimes combined; several good examples of this are to be found in New York residences.

The Hennebique system has been introduced in several constructions of this kind.

Then domes and arched roofs are occasionally constructed in this material.

Two notable examples may be referred to, namely, the dome of a cathedral at Poti, Russia, and that of the New Wesleyan Methodist Hall, Westminster. Illustrations of one, and a brief description of both of these are given.

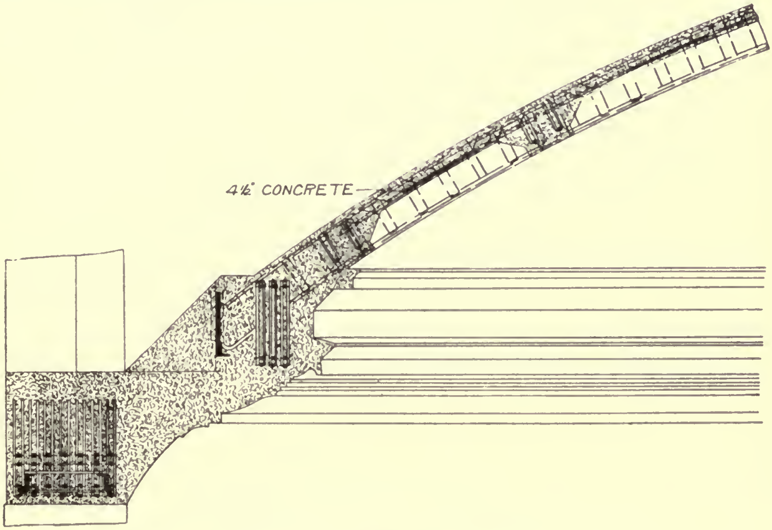


FIG. 307.

Reinforced Concrete Cathedral at Poti, Russia.—The dome of this cathedral has been built on the Hennebique system. The whole building is in reinforced concrete, and has been fully described in "Concrete Engineering" for August, 1910, published in Cleveland, U.S.A.

A few particulars will be given regarding the construction of the dome only. The reinforcement of this consisted of tension and compression bars, and of stirrups in the ribs, the slabs between these ribs being reinforced by wire mesh reinforcement; these slabs were 4 in. in thickness, and the ribs 10 in. deep on the outside. The space between the ribs was filled in with an insulating material, and the surface of the dome was covered with sheet iron.

Dome of New Wesleyan Methodist Hall.—This is illustrated by the drawing, Fig. 307 and is a good example of what can be done in reinforced concrete. The work is being carried out on the Kahn system, the reinforcement consisting of Kahn bars of various sections, but chiefly $1\frac{1}{4}$ in. T.'s, with angle irons, and metal plates of circular shape.

This important work was designed by Messrs. Lanchester and Rickards, Architects, London.

Roofs of Sawtooth Shape.—A number of roofs of this class have been erected in reinforced concrete, and the material is eminently suitable for their construction.

Expanded Metal in Arched Roof Construction.—A good example of the use of expanded metal in arched-roof construction, occurs in the roof of St. Barnabas Church, Dalston, London.

The views show the nave roofing in progress, and also the temporary timbering to dome.

The architect for this work was Prof. C. H. Reilly, M.A., A.R.I.B.A., of Liverpool.

Illustrations of reinforced concrete flat roofs have already been given.

Stair Construction.—Stairs are constructed in this material in a variety of ways.

One of the authors is now constructing at Bridlington two flights of steps, each flight 8 ft. in width, and these are reinforced in the manner shown on the accompanying section.

A 1 : 2 : 3 mixture is being used, namely, 3 of gravel, capable of passing through a 1 in. ring, and 2 of clean sharp sand, to 1 of cement.

The reinforcement consists of $\frac{3}{8}$ in., $\frac{1}{2}$ in., and $\frac{5}{8}$ in. bars and patent clips.

The steps have moulded treads, and are finished in non-slip carborundum concrete.

The authors have described in the Chapter on General Engineering, the Stand at the York Race Course, where the various flights of steps, in fact the whole structure has been built in reinforced concrete, and should prove of interest to the reader.

At the Stoke-upon-Trent Town Hall Extensions the rises in the gallery are formed in reinforced concrete (see Fig. 311).

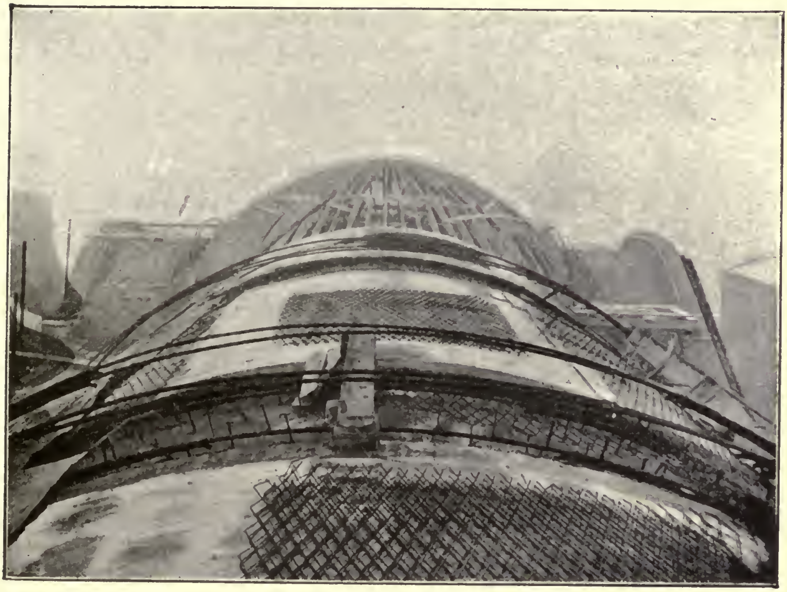


Fig. 308.

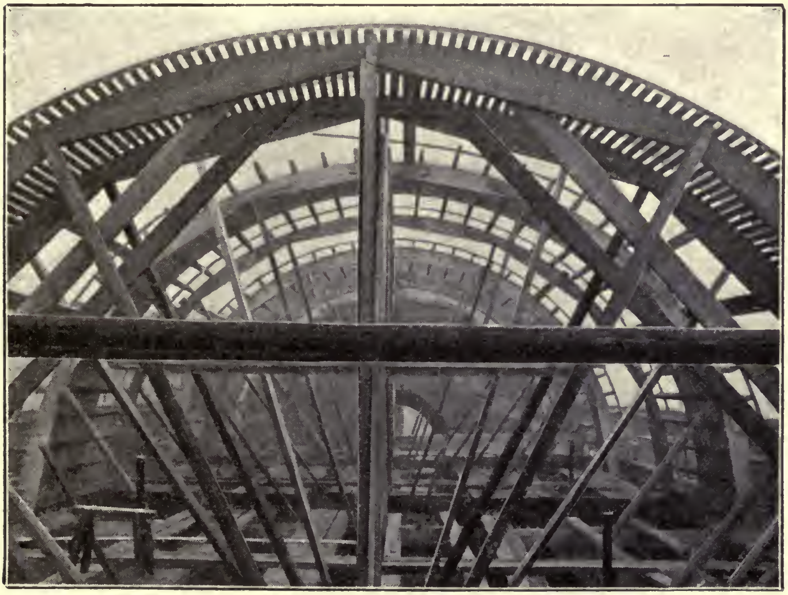


Fig. 309.

The illustrations are self-explanatory.

Many staircases have also been constructed on the Lock

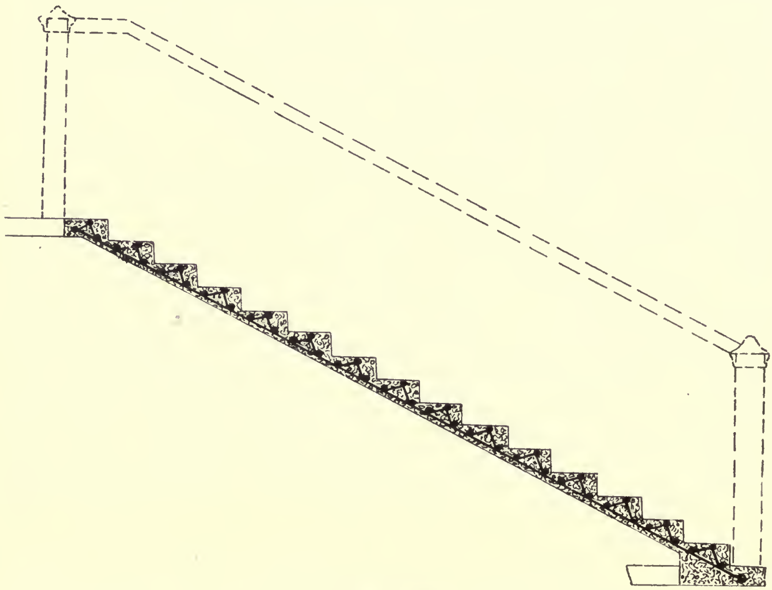


Fig. 310.

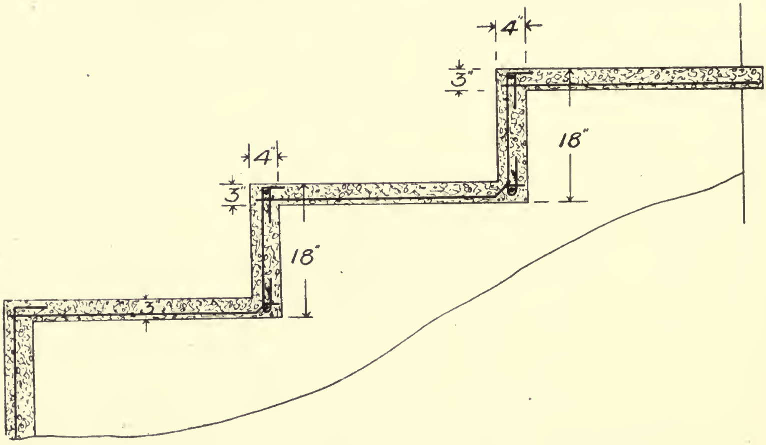


Fig. 311.

Woven Mesh System, the two methods usually adopted being shown in Figs. 312.

Balconies and Cantilever Platforms. — These are often constructed in this material. The authors have already described in the chapter on Municipal Engineering, the cantilever platform forming the Fish Stand recently constructed at Bridlington. A good many other examples might have been given, particularly of works on the Hennebique System.

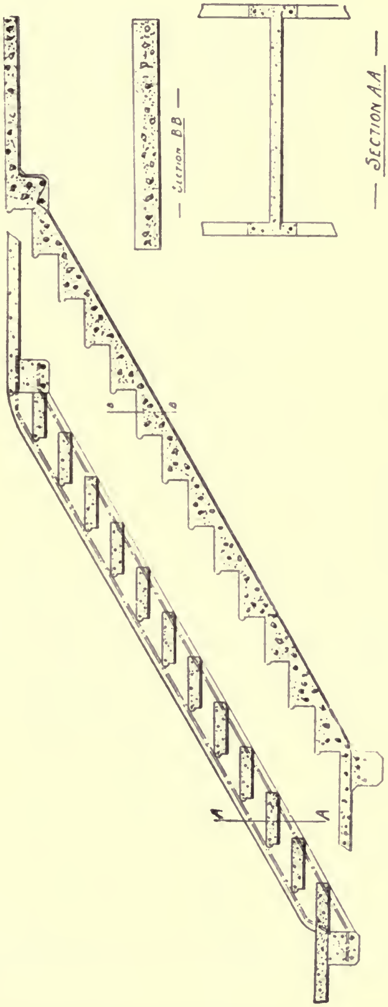


Fig. 312.

A balcony recently constructed at the Girls' High School, Bridlington, is shown in Fig. 313. It is an interesting piece of work, and was well tested before use.

It was built on the "Wells' System of reinforcement by Stuart's Granolithic Co., Ltd., the Architects being Messrs. Botterill, Son & Bilson of Hull.

The Tallest Reinforced Concrete Building in the World.—This is illustrated in Figs. 314 to 320 and is known as Ingall's Building at Cincinnati, Ohio, U.S.A. The building is a sixteen-storey one, and has been fully described in "Engineering News," 30 July, 1903, in which some of the illustrations here given have appeared.

It is a remarkable structure, and the views show the building at various stages of its erection.

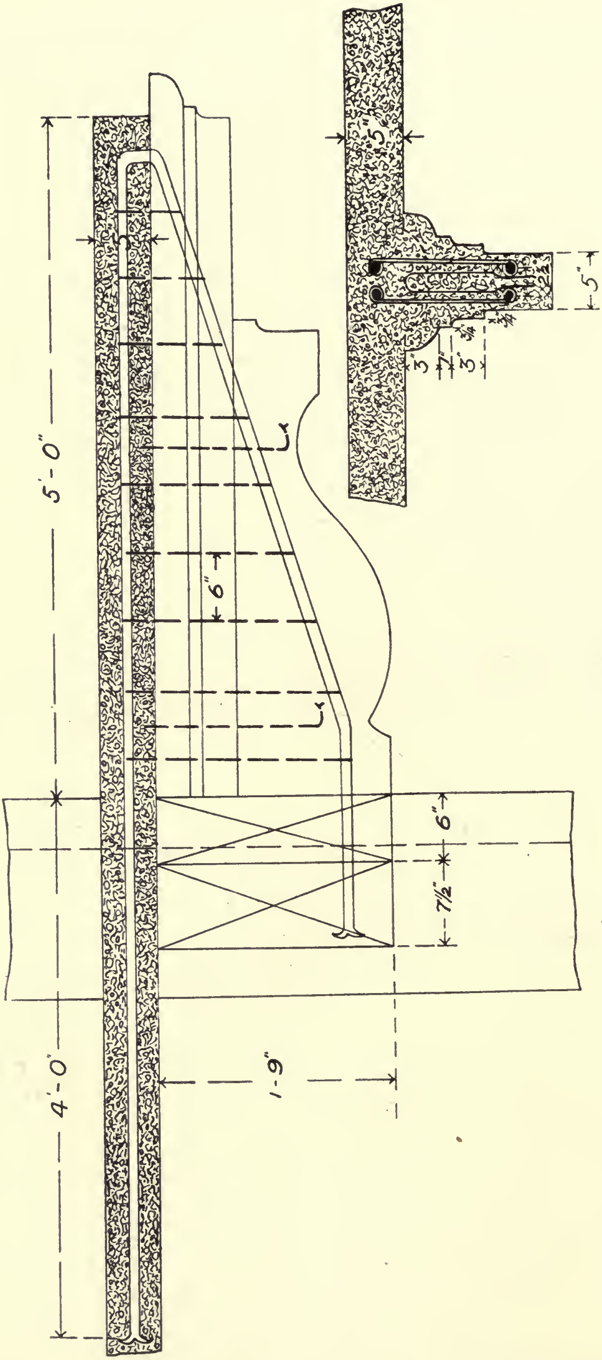


FIG. 313.

Fig. 314 shows the lower six storeys in course of erection.
Fig. 315 shows the construction of one of the floors.

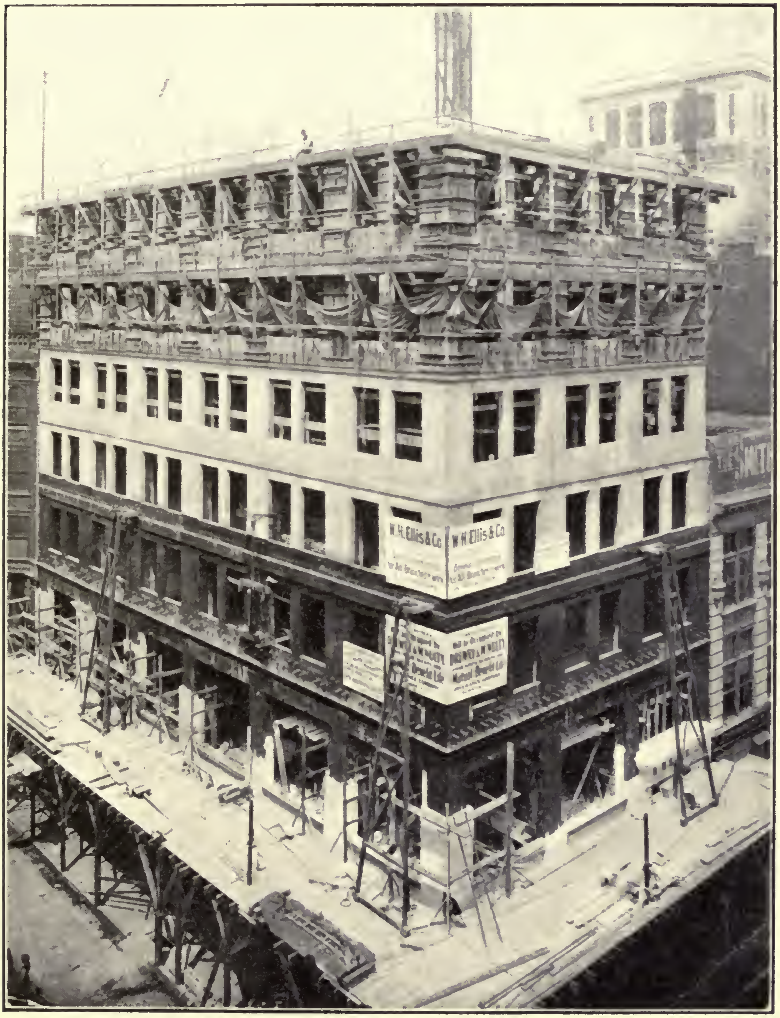


Fig. 314.

Fig. 316 shows the construction of another floor.
Fig. 317 shows some of the beams and columns.

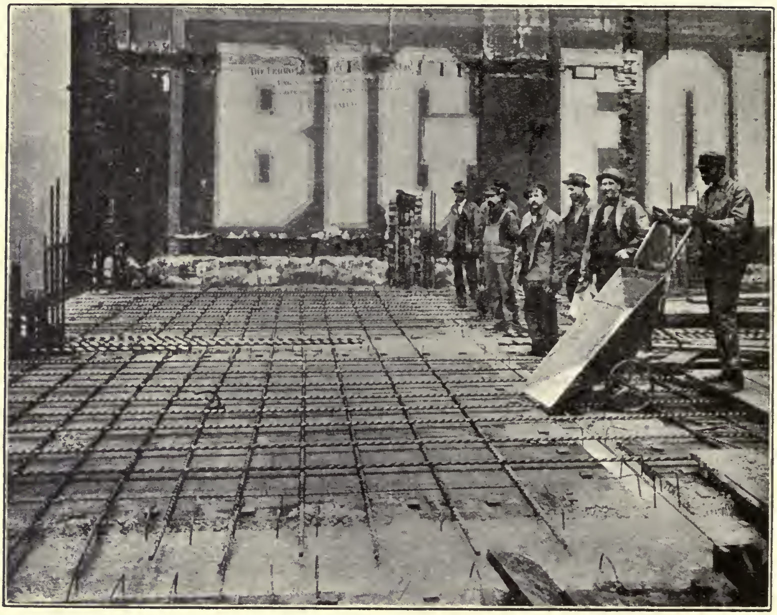


Fig. 315.

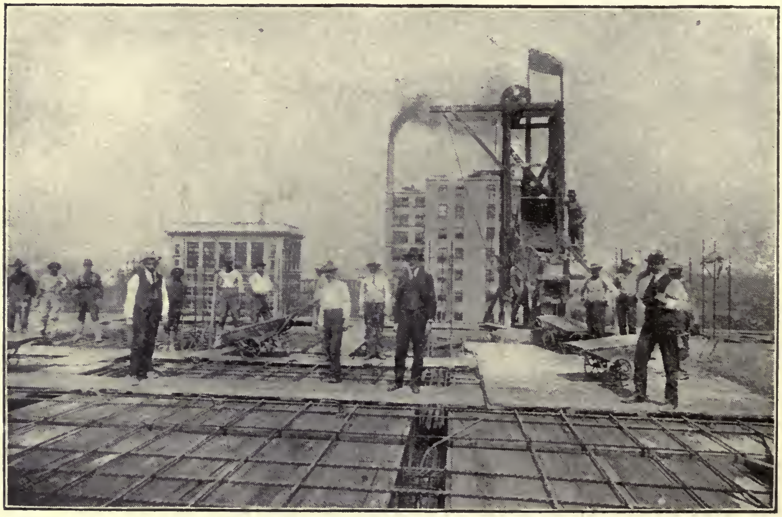


Fig. 316.

Fig. 317.

Fig. 318.

Fig. 318 shows other beams and columns.

Fig. 319 shows the building nearing completion.

Fig. 320 shows the completed structure.

The engineers and contractors for this interesting building were the Ferro-Concrete Construction Co., of Cincinnati. The building is situated at the corner of Vine and Fourth Streets, and is owned by the Ingall's Realty Co.

The first floor is occupied by railway ticket and telegraph offices, stores, and entrance hall; the second by a bank; the whole of the top floors being in the occupation of the Western Union Telegraph Co.

The Architects were Messrs. Elzner & Anderson of 18 E. Fourth Street, Cincinnati.

The reinforcement is on the Ransome System, and consists of rods, stirrups, and hoops of twisted steel. Plain bars were used in the columns.

The general contractors were Messrs. W. H. Ellis & Co., Cincinnati. The work was commenced in October 1902 and completed by the end of 1903.

The building occupies a site, the dimensions of which are approximately 100 ft. by 50½ ft., and the natural foundation consists of gravel and sand.

The building has four hydraulic passenger elevators. The exterior face-work to a height of three storeys is of 4½ in. marble; above this it is of glazed light-grey brick with terra-cotta trimmings.

The columns are 16 to 33 ft. apart, centre to centre, and vary in size from 34 by 38 in. at the bottom to 12 by 12 in. at the top.

Fig. 319.

FIG. 320.

CHAPTER XI.

GENERAL NOTES.

THE authors propose in this concluding chapter to deal with many matters which occur in the actual carrying out of reinforced concrete work. Such points will be dealt with as, the finishing of concrete surfaces, the waterproofing of concrete, bonding of old and new concrete, effects of sewer gases on concrete, choice of concrete mixers, expansion joints, breeze concrete, action of alkali on Portland cement, the prevention of failures, erection and removal of forms, and other important matters.

1. **Experienced Designer.**—They would first emphatically state that as reinforced concrete work is a speciality, the designing of structures to be erected in this material should be entrusted to those who have had experience with the material; it is not sufficient for an engineer or architect to insert into his concrete slab or beam a certain amount of reinforcement, the amount being often a matter of guess-work, a thorough knowledge of what the slab or beam which has been designed is capable of doing is absolutely necessary.

2. **Skilled Workmen.**—And not only should the designer be a man who thoroughly understands his work, but it is equally important that the placing in position of the reinforcement, and the supervision generally of the work, shall be entrusted to a competent foreman who has had experience in this class of work, and should not be left in the hands of an ordinary concreter or bricklayer, as is so often the case.

3. **Erection and removal of Forms.**—There are matters that require the greatest care. In connection with all reinforced concrete work the erection of the centres and forms is a large item compared with the total cost of the works. The centering should be of such dimensions, and so constructed, as to

remain rigid during the laying and punning of the concrete. It should be so arranged as to permit of its easy removal. Provision should also be made wherever practicable for splaying or rounding the angles of the concrete. It is advisable to lime-wash the centering before the concrete is filled in.

The authors give several illustrations of how forms should be constructed. Fig. 321 represents the method of building forms for a 2 ft. sq. column. The angles are not splayed in this case.

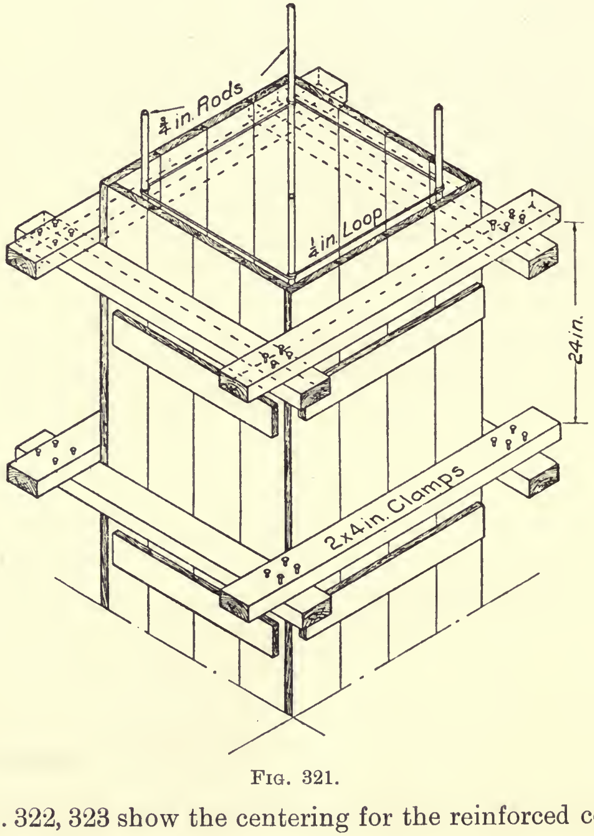
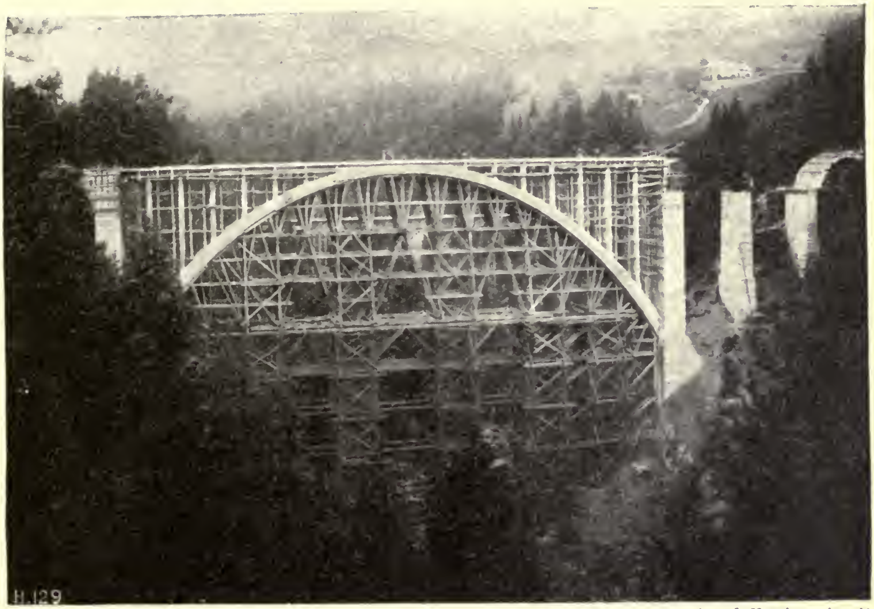


Fig. 321.

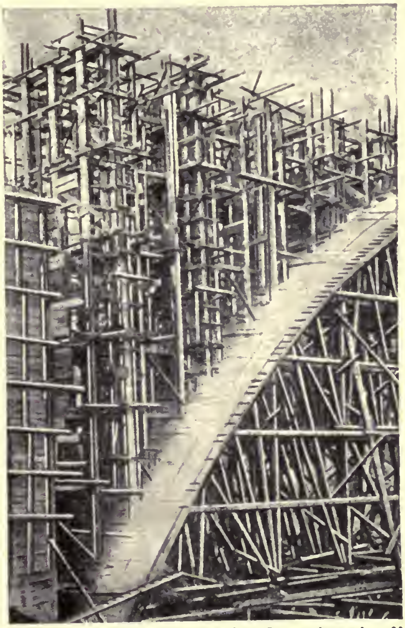
Figs. 322, 323 show the centering for the reinforced concrete bridge near Teufen, Switzerland.

Fig. 324 shows the timbering for the new retaining wall for the Kensington Borough Council designed and carried out by Leslie & Co., London.



"Concrete and Constructional Engineering."

FIG. 322.



"Concrete and Constructional Engineering."
FIG. 323.

The removal of the forms should be done in a very careful manner. On no account should they fall with a crash, but should be taken down timber by timber. The main supports should always be left to the last.

4. **Waterproofing of concrete.**—Large numbers of experiments have been made from time to time to determine if possible the best means of rendering concrete watertight, that is, when the concrete is under water pressure on one side.

In America the following method is often adopted.

Liquid asphalt is mopped on to the concrete, this forms a coat upon the surface of the unset concrete, the thickness of the asphalt varying from $\frac{1}{4}$ to $\frac{1}{2}$ in. This method is open to the objection that asphalt when exposed to the action of water has a very short life, and the authors do not recommend it.

FIG. 324.

Some engineers in America adopt the following practice:—They have the face of the reservoir wall or dam rendered over with a semi-liquid mortar, composed of:—

1 part Portland cement.
3 parts sand.
$\frac{1}{2}$ part thoroughly slaked lime.

When this is set, the face of the wall receives a coat of cement grout.

The method recommended by Prof. Ira. O. Baker[1] consists of the application to the concrete of a compound of alum and soap. This may give satisfactory results in a laboratory, but the authors do not think it can be relied upon in actual work.

[1] "The Technograph and Contractors' Record," 23 Feb. 1910, p. 480.

"Watertight" concrete in their opinion can be obtained where a most careful selection is made of the aggregate, the latter being small, able to pass through a $\frac{3}{4}$ in. ring; where the greatest care is taken in mixing and placing the concrete, and where a fairly "wet" mixture is put in. On no account should a "dry" mixture be made if intended to be watertight.

5. **Expansion joints.**—These are very necessary in walls of large sectional area and considerable length. Several examples of the insertion of expansion joints have been given by the authors, notably the illustrations of the Angel Road Railway Bridge, and the Bridge at Thierry.

Undoubtedly many of the cracks which one observes in long walls of concrete would not have occurred had there been a careful distribution of expansion joints, which really locate the cracks to predetermined positions.

6. **Concrete Mixers.**—Wherever works of any magnitude are to be carried out, a concrete mixer should be used, and in selecting a machine one should be chosen into which the materials, including the water, can be accurately proportioned, and which will produce a mixture of uniform consistency.

There are many good machines in the market,[1] some of the best known being, the, Owens Gravity Mixer, Koehring Mixer, Pansy Mixer, Kent Precision Mixer, Taylor Mixer, Coltrin Mixer, Empress Mixer, Trump Mixer, Universal Drum Mixer, Smith Mixer, Ransome Mixer.

A "batch" mixer is preferable to a "continuous" mixer, as the proportions are more likely to be correctly maintained.

7. **Finishing of Concrete Surfaces.**—The finishing of a concrete surface may at first seem a matter of little consequence, but it is really one of great importance, and requires the most careful consideration. Many English engineers favour the rendering with cement mortar (1 of cement to 2 of sand) of the surface of the concrete to about an inch in thickness. American engineers on the other hand deprecate this practice, believing that it is impossible to get cement rendering to per-

[1] Vide "Concrete and Constructional Engineering," Jan. 1909 and May, 1910.

manently adhere to a concrete face, and the authors admit that there is a great deal to be said for such a contention.

A method which the authors advise is as follows :—Assuming that the moulds are well constructed (planed boards being used) and perfectly clean ; also that the concrete has been carefully placed ; the slight imperfections in the face of the forms, and the joints between the boards which have left their marks upon the plastic concrete, can be easily removed when the mixture has hardened by rubbing down with sand paper after taking off the projections with a chisel. Any small cavities should next be filled with cement. When this has set the whole face should be washed down with a thin grout of the consistency of whitewash, mixed in the proportions of 1 part of cement to 2 parts of sand, the wash being applied with a brush.

This method the authors consider is far preferable to that of cement rendering.

If the finished work is inside a building, and it is necessary that it should present a very neat appearance, the following is a good method to adopt: Dress down the face of the wall or underside of ceiling, or the sides of the beams, and apply a thin coat (less than ½ in.) of Parian Cement; this makes an excellent finish.

8. **Colouring Matter.**—This is used a good deal in the United States, but not much in this country. The cement, sand, and colouring matter are mixed together dry, and it must not be forgotten that the mortar will appear several shades darker while wet than after it has dried.

By mixing five pounds of colouring-matter with a bag of cement the following colours are obtained [1] :—

Raw iron oxide will give bright red.

Roasted iron oxide will give brown.

Ultramarine will give bright blue.

Yellow ochre will give buff to yellow.

Carbon black or lamp-black will give grey to dark slate.

Manganese dioxide will give black (using eleven pounds per bag of cement).

It should be noted that in all cases the addition of mineral

[1] "Concrete and Constructional Engineering," Dec. 1910, p. 904.

colours causes a loss of strength, but this is not important seeing that the colour is used only in the surface coat. Lighter shades may, of course, be obtained by using one-half the quantities above specified.

9. **Pebble-dash Finish.**—This method is sometimes adopted, and the finished work presents a very pleasing appearance. The work should be screened from the sun and kept wet for three or four days.

10. **Breeze Concrete.**—At the present time a great deal of discussion is going on regarding this. It is the authors' opinion, however, that slag, clinker, and coke-breeze are not suitable aggregates for reinforced concrete. The compressive strength of concrete formed of such aggregates is not satisfactory, and cannot compare with concrete in which gravel or broken stone has formed the aggregate.

There is also a risk of such an aggregate having a chemical effect upon the reinforcement. The sulphur in the breeze undoubtedly has also a deleterious effect upon the concrete, and will cause such concrete to become disintegrated.

Mr. E. P. Wells carried out an interesting experiment with breeze concrete some little time ago, which he describes as follows [1] :—

"The experiment was with a beam of 14 ft. span by about 4 ft. wide and 6 in. thick. Half-inch rods for reinforcing were used with about $\frac{3}{4}$ in. of concrete covering them as a minimum. The rods were clean when put in; but after the lapse of twelve months, when the beam was broken, the rods were badly oxidized, and deep pittings had taken place in all of them. There are many instances of this kind" (Mr. Wells goes on to say) "and I strongly advocate that in no case whatever should breeze or cinder concrete ever be used where small rods for reinforcing purposes are inserted."

11. **Avoidance of Stone Dust.**—Stone dust should never be allowed to form part of the aggregate of concrete. The dust particles are inert and entirely without value, and if broken stone forms the aggregate, it should be well screened before it is used.

[1] Lecture by E. P. Wells on "Concrete and Reinforced Concrete" (L.C.C. School of Building at Brixton), February, 1910.

12. **Roughening a Concrete Surface.**—It is occasionally required that the finished surface of a floor shall present a roughened appearance. This is easily done by the use of what is known as a bush-hammer. This tool has a wide striking face with several rows of projecting points, by means of which the concrete is roughened.

13. The material required for one cubic yard of concrete is so often needed by engineers that a table setting it out is very useful; such a table is given in "How to use concrete," published by the Concrete Publishing Co., of Detroit, Mich., U.S.A., from which the following is taken.

Concrete with gravel ¾ in. and under.

Proportions required for one cubic yard concrete.					
Cement.	Sand.	Gravel.	Cement. Bushels.	Sand. Cubic yards.	Gravel. Cubic yards.
1	1	2	2·72	0·41	0·83
1	1	2½	2·41	0·37	0·92
1	1	3	2·16	0·33	0·98
1	1½	2½	2·16	0 49	0·82
1	1½	3	1·96	0·45	0·89
1	1½	3½	1·79	0·41	0·96
1	1½	4	1·64	0·38	1·00
1	2	3	1·78	0·54	0 81
1	2	3½	1·66	0·50	0·88
1	2	4	1·53	0·47	0·93
1	2	4½	1·43	0·43	0·98
1	2½	3½	1·51	0·58	0·81
1	2½	4	1·42	0·54	0·87
1	2½	4½	1·33	0·51	0·91
1	2½	5	1·26	0·48	0·96
1	2½	5½	1·18	0·44	0·99
1	3	4	1·32	0·60	0·80
1	3	4½	1·24	0·57	0·85
1	3	5	1·17	0·54	0·89
1	3	5½	1·11	0·51	0·93
1	3	6	1·06	0·48	0·97
1	3½	5	1·11	0·59	0·85
1	3½	5½	1·06	0·56	0·89
1	3½	6	1·00	0·53	0·92
1	3½	6½	0·96	0·51	0·95
1	3½	7	0·91	0·49	0·98

14. **Acetic Acid and Concrete.**—Though one of the weakest

acids, acetic acid is very violent in its action on concrete. Concrete tanks designed for containing vinegar should be lined with glass, or other material which will not be affected by the action of acetic acid.

15. **Action of Alkali on Portland Cement.**—It has been stated that the alkalies of our soils are destructive to Portland cement, and that this harmful action, which is a chemical one, is going on most rapidly in partly submerged work such as piers or dams. The experiments of Prof. Edwin Burke and Mr. E. Tappan Tannatt [1] would seem to substantiate this. The authors, however, do not consider that this matter has been sufficiently investigated to warrant them in expressing any opinion thereon The matter is still in its experimental stage.

16. **Action of Sewage and Sewage Gases on Concrete.**—In an interesting paper on this subject read in April, 1910, by Mr. Sidney H. Chambers, before the Concrete Institute, it was pointed out that under certain conditions disintegration of concrete was caused by sewage and sewage gases, and the conclusions he arrived at were as follows :—

"That the gases in solution in sewage, and those expelled from it, arising from its decomposition, do act injuriously upon Portland cement concrete, notwithstanding the fact that the concrete is constituted of sound and good materials, when the following conditions prevail :—

"(1) A high degree of putrescence of the sewage.

"(2) A moistened surface, which held or absorbed the putrid gases.

"(3) The presence of a free air supply.

"Further, that in the absence of one or the other of the above enumerated factors little danger from erosion need be feared."

17. **Bonding of old and new Concrete.**—This is a matter that requires the greatest care, as shrinkage cracks almost invariably occur in concrete floors, etc., along the joints between successive layers of concrete. Even retaining walls will occasionally give evidence where one day's work ended and the next began, and frequently a crack will occur along this line. Messrs. Taylor

[1] "Action of Alkali on Portland Cement," by E. Tappan Tannatt in "Concrete Engineering," Cleveland, U.S.A., May, 1910, p. 120.

and Thompson in their "Concrete, Plain and Reinforced," page 376, cite an interesting case in the New York Subway; they say:—

"Work was commenced with no provision for bonding horizontal layers, but it was soon found that more or less seepage occurred, and in one case where a large arch was torn down the division line between two days' work was distinctly seen."

How then should bonding of the old and new concrete be effected? The authors recommend the following method:—

Sweep off the concrete surface thoroughly with a stiff broom, applying water, none of which must remain upon the concrete. As soon as the excess water has been taken up by the atmosphere or absorbed by the concrete, well grout the whole surface with a sand and cement (2 to 1) grout, put on $\frac{1}{2}$ in. in thickness. Repeat this after an interval of a few minutes; then proceed with the next layer of concrete.

18. **Effects of Sea Water on Concrete.**—One of the authors (Mr Matthews) has carried out a series of tests in order to ascertain the effects of using sea water in the mixing of concrete, and the result may be stated briefly to be that sea water has no immediate detrimental effect upon the tensile strength of the concrete, for at the expiration of 7 and 14 days from the mixing of the concrete, it will be quite as strong as if it had been mixed with fresh water; at 28 days, however, it will be of considerably less tensile strength. His results are set out in the following table:—Briquettes mixed with sea water and immersed (after 24 hours under damp flannel) in sea water.

Mixed with sea water.			Mixed with fresh water.		
Days.	Neat.	3 to 1.	Neat.	3 to 1.	Remarks.
7	693	180	685	200	Same consignment of cement used in both cases.
14	775	287	787	277	
28	773	293	875	353	

The initial set when mixed with fresh water was 35 minutes, when mixed with sea water 9 minutes; permanent set, fresh water 6 hours; sea water 6 hours, so that we observe that when concrete is mixed with sea water its initial set occupies about one-fourth the time that it does when mixed with fresh water, the time for the permanent set being the same. The authors, as already stated in Chapter I, do not recommend the use of sea water except for mass concrete in marine works.

With regard to the action of sea water on concrete permanently immersed in same, or alternately in and out of sea water as occurs in tidal works, the most important tests which appear to have been carried out in order to ascertain this, are the Scandinavian and German Tests described in "Concrete and Constructional Engineering," January, 1910, p. 23. The conclusions arrived at were as follows:—

"(1) Good Portland cements, such as are now on the European market, are very resistant to the action of sea water. A marked difference in the behaviour of cements of slightly different composition has not been found, except that a high proportion of aluminates tends to cause disintegration.

"(2) In a dense mortar, the chemical action is confined to an outer layer of small depth, further action being checked by the slowness of diffusion. A porous mortar, by admitting salt-water to the interior, is apt to crack by expansion owing to chemical change.

"(3) The main agency in the destruction of mortar and concrete in marine embankments, harbour works, groynes, etc., is not chemical action, but the alternations of saturation, drying in the sun, freezing, etc., due to the alternate exposure and covering by the rise and fall of the tide. Destruction takes place sometimes by cracking, sometimes by scaling, the latter effect being produced especially by frost.

"(4) The denser the mortar the better (1 cement : 3 sand is too poor). An admixture of fine sand with the ordinary sand increases the closeness of the mixture. A well-graded aggregate would be advantageous for the same reason.

"(5) The addition of finely ground silica or trass to the cement before mixing is possibly advantageous in the case of weaker mortars. It is very doubtful whether anything is gained by adding trass to the richer mortars.

"(6) Hydraulic lime mixed with trass, etc., whilst of some value, where a cheap material is required, in the mild climate and absence of tide of the Mediterranean, is incapable of withstanding the conditions of coast work in northern latitudes.

"(7) The destructive action of the sea being mainly physical and mechanical, and not chemical, tests by mere immersion in still sea water are of very little value in determining the behaviour of concrete in marine engineering works. A mixture which disintegrates under this test is certainly useless, but a mixture which passes the test may disintegrate under the more stringent conditions of practical use.

"(8) As long a period as is practicable should be allowed for the hardening of concrete blocks before placing in the sea. The German recommendation of one year in moist sand before setting in place is probably impracticable in most places, but should be approached as nearly as possible.

"(9) The behaviour of test specimens for the first twelve months is very irregular, and definite conclusions can only be drawn from the results of long-period tests.

"Both of the reports contain very full tables of crushing and tensile tests, and a complete record of the appearance of each concrete block at stated intervals."

19. **The Prevention of Failures.**—Failures in reinforced concrete structures may be divided into two classes.

(1) *Failures from unavoidable causes.*—Under this class will come earthquakes, inundations, lightning, fire, tempest, explosions; and the authors know of no material which, if properly designed, can resist any of the above so well as this material.

(2) *Failures from preventable causes.*—Among these may be included failures owing to errors in calculation, insufficient or badly arranged shuttering, mistakes in the construction of the pillars, a poor concrete mixture owing to unsuitable aggregate or cement, or both, being used, or through bad workmanship or inefficient supervision.

Any of the above causes may be responsible for a reinforced concrete structure being a failure, but there is absolutely no reason, if these matters are carefully attended to, why every structure should not be a *complete success*.

20. **Test Loads.**—A test load is frequently applied before a reinforced structure is used ; it is not necessary with every structure, but with bridges, floors, balconies, and similar works the authors look upon it as a desirable safeguard.

INDEX.

D.

E.

F.

G.

H.

I.

J.

K.

L.

M.

S.

T.

U.

V.

W.

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