CONCRETE

AND

REINFORCED CONCRETE

CONSTRUCTION

BY

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PREFACE.

The marvelous growth of the cement industry during the past few years has led to the present time being spoken of as the cement age. The use of cement concrete in many forms of construction for which heretofore other materials have been used has created a demand for concise and reliable information in regard to the use of concrete. It has been the author's aim in the preparation of this book to make it, as far as possible within reasonable limits, a complete treatise on the properties and uses of concrete and reinforced concrete, as applied to construction. As far as is possible, a logical development of the subject has been followed. The book is not only intended as a reference work for engineers, architects and contractors, but, it is believed, the treatment is sufficiently simple for the engineering student and general reader. Clearness in the development, and, as far as possible, a continuity of treatment of the subject matter has been attempted, at the risk, perhaps, of some repetition.

In the early chapters a discussion of the materials used for concrete has been given. While perhaps a knowledge of how cement is manufactured is not necessary to doing good work in concrete, it certainly can do no harm, and it is believed by the author that the more that a cement user knows about the material with which he is working the less danger there is of his abusing it. Following the discussion of cement, the aggregate is taken up for consideration; the different mixtures used are considered, together with the effect of size of sand, gravel and broken stone, and the effect of impurities. Proportioning concrete for different uses is next considered, and the methods of determining the voids briefly discussed. The methods used for both hand and machine mixing are described. The various types of machine mixers are discussed, and a machine of each type described. A number of examples of mixing plants are given. In Chapter VI., on Placing Concrete, the use of grout and of rubble concrete is briefly discussed. The tools for mixing, conveying and ramming concrete are described, and the methods of laying and protecting concrete in freezing weather are described. The various methods of depositing concrete under water are also considered. In Chapter VII., on Cost of Concrete, no attempt is made to give a large number of cost data, but rather it has been the author's purpose to analyze the factors entering into cost, so that the estimator may form a correct cost estimate from known conditions.

Chapter VIII. describes the various methods of finishing and treating concrete surfaces, together with methods of making ornamental mouldings, etc. The methods of coloring concrete are also described.

Chapter IX. discusses the effect of freezing on concrete, tells how to secure an impermeable concrete, and describes various methods of waterproofing. The effect of sea water on concrete and the effect of oil on cement and concrete are discussed. The preservation of metal in concrete, adhesive coefficient of expansion and fire resisting qualities of reinforced concrete and effect of flue gases are also discussed.

Chapter X. treats of the strength and elastic properties of concrete. The various elements affecting these properties are also given, and the results of numerous tests are quoted.

Chapter XI. treats of the reinforcing metal. In Chapters XII. to XVII. the principles and disposition of the reinforcement are discussed. The methods devised to secure mechanical bond, together with various styles of reinforcements used for slabs, beams, columns, walls, arches and pipes, are described and illustrated.

Chapter XVIII. treats of the general phenomena of flexure. The action of a beam under tests and when tested to failure are discussed, together with the various stresses developed. The results of the latest tests are freely quoted in this chapter.

Chapter XIX. gives a clear and concise exposition of the theory of beams, while various beam theories used and proposed are set forth in Chapter XX.

Chapter XXI. gives the theory of columns with both straight and hooped reinforcement. Working formulas are given, and results of latest available tests quoted.

Chapter XXII. discusses the bearing power of soils, spread and pile foundations, and gives a large number of examples of foundations actually built.

Chapter XXIII. discusses the application of reinforced concrete to building construction. Columns, floor slabs between beams, monolithic floors, arch floor construction, walls, partitions, roofs and stairways are taken up, described and illustrated, and examples from actual work given. In Chapter XXIV. the practical construction of buildings is taken up. Sheathing and centering are discussed. Illustrative examples of forms for columns, floor, girders, roofs and walls are given.

Chapter XXV. is devoted to the discussion of retaining walls, also the expansion and contraction of concrete due to setting and thermal changes. Numerous examples of retaining walls of T-section and of the counterfort types are given.

In Chapter XXVI. the application of reinforced concrete to the construction of dams is discussed, and illustrative examples of types to be used under varying conditions given.

In Chapter XXVII. the application of concrete, both plain and reinforced, to the construction of sewers and conduits is taken up. European and American methods for the manufacture of cement pipe are given. Numerous examples are given of sewers and conduits, particular attention being paid to the methods employed by American engineers.

Chapter XXVIII. is devoted to tank and reservoir construction. The application of reinforced concrete to stand pipes and water towers is discussed. Its application to reservoir construction is illustrated by a number of well-known American reservoirs. Its use for grain elevators, sand storage bins, coal pockets and gas holder tanks is taken up.

In Chapter XXIX. the application of reinforced concrete to chimney construction is illustrated by a number of examples. Its use in tunnel and subway construction and for railroad ties, fence posts, piers and wharfs is also considered.

Chapter XXX. is devoted to bridge construction. Girder bridges of various types are considered, and examples of the different types given. Arch bridges of concrete, with and without reinforcement, are also considered, and numerous examples given. The subject of culvert construction is also amply illustrated in this chapter.

In Chapter XXXI. the subject of forms and arch bridge centers is discussed.

Chapter XXXII. illustrates the application of reinforced concrete to the construction of bridge floors. Chapter XXXIII. shows the application of reinforced concrete to bridge piers and abutments.

Chapter XXXIV. is devoted to concrete blocks. The various types used, and their application to building construction, are briefly described. Almost without exception illustrative examples of the application of concrete to the various forms of construction have been taken from actual practice.

The author is indebted to the following firms for drawings and information in regard to structures described: H. C. Miller & Co., Tucker & Vinton, Unit Concrete Steel Frame Co., Reinforced Cement Construction Co., Concrete Steel Engineering Co., Trussed Concrete Steel Co., American Concrete Steel Co., Monolithic Steel Co., The Foundation Co., Clark & Co., Raymond Concrete Pile Co., Weber Steel Concrete Chimney Co., Wilson-Baillie Manufacturing Co., the Ambursen Hydraulic Construction Co., St. Louis Expanded Metal and Corrugated Bar Co., and the Ransome Concrete Machinery Co.

In the preparation of this book the author has drawn freely from both European and American engineering journals, and from the proceedings of various technical societies. In fact, in order to keep up with current practice, it is necessary to read one or more of the excellent engineering journals now published. The author is especially indebted to the "Engineering News," "Engineering Record," "Railroad Gazette," "Engineering-Contracting," "Cement," "Cement Age," "Cement and Engineering News," and the Proceedings of the American Society of Civil Engineers and Western Society of Engineers.

Among the works consulted were the following: Watertown Arsenal Tests; Reports of Chief Engineers, U. S. A.; Patton's Foundations; Baker's Masonry Construction; U. S. Government Reports; Rafter's Tests; Eckles' Cements, Limes and Mortars; Concrete, Plain and Reinforced, by Taylor and Thompson; Reinforced Concrete, by Marsh; Cement and Concrete, Louis C. Sabin; Reinforced Concrete, Buel and Hill; Cements, Mortars and Concretes, M. S. Falk; Beton Armé, Christophe; Practical Cement Testing, W. Purves Taylor; Gillette's Hand Book of Cost Data, and the writings of Dr. Michaelis and M. Considère.

Much valuable information in regard to tests being conducted at various engineering schools has been furnished the author by

Profs. Swain, McKibbon, Spofford, Turneaure, Howe, Hatt, Woolson and Talbot. The author is also indebted to Messrs. Spencer B. Newbury, Edwin Thacher and E. P. Goodrich.

Although the increase in the use of concrete, both with and without reinforcing metal, has become so great in the past few years as to almost warrant it being said that we are going concrete mad, it should be remembered that reinforced concrete does not possess wonderful and mysterious properties such that it may be unscientifically or recklessly used. To the contrary, it should be used with the same care and judgment that has made other and older kinds of construction both safe and satisfactory. It should also be remembered that there are other and tried kinds of construction which are much more suitable for use in many situations. Under such conditions enthusiasm for a given form of construction should be tempered with good judgment, and the most suitable building material chosen. Unless this is done the ethics of good engineering will be violated.

HOMER A. REID.

January 15, 1907.

PREFACE TO THE SECOND EDITION.

The favor with which "CONCRETE AND REINFORCED CONCRETE CONSTRUCTION" has been received by engineers and others interested in the subject having made necessary a new printing, the author has taken advantage of the opportunity to correct certain typographical errors and to make such additions as were necessary to bring certain portions strictly up to date. A number of new forms of reinforcing bars and frames have been developed and have come into use since the book was first printed and illustrations and descriptions of these have been added. In a similar manner other sections have been changed and amended to bring the text fully up to the latest developments in concrete and reinforced concrete construction.

H. A. R.

February, 1908.

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Concrete and Reinforced Concrete Construction

INTRODUCTION.

Use of Cement by Egyptians and Romans.—The recent marvelous growth of the cement industry, due to the wide use of concrete in construction, has led to its being spoken of as a new industry. Yet hydraulic cement has been used since the dawn of civilization. It is known that the Egyptians, 4,000 years ago, made a natural cement which set under water. While Carthage was at the height of her glory, some 2,300 years ago, an aqueduct over 70 miles in length was built to furnish a water supply for that ancient city. Natural cement was used in its construction. To cross a valley, over 1,000 arches were built. Many of these were over 100 feet high, and some are still standing. Cummings, in his "American Cements," states that at one point a piece of masonry over 100 feet long has fallen from the top of the aqueduct to the rocks below and still lies there intact, unbroken, illustrating the toughness, tenacity and durability of the natural rock cement used by these early constructors.

The Romans used hydraulic cements of such good quality for the construction of sewers, water mains, foundations, buildings and roads that relics possessing great strength and toughness are to be seen at the present day. The dome of the Pantheon, erected at the beginning of the Christian era, is perhaps the largest example of concrete construction coming down from the ancients. This magnificent structure, which is 142 feet in diameter, and contains a 30 foot opening at the top, has withstood the destructive elements of time for 19 centuries, and to-day does not show a single crack. It is stated that in Mexico and Peru, natural rock cement was used so long ago in stone masonry, that the stone has worn away, leaving the projecting mortar joints.

Smeaton's Rediscovery of Hydraulic Cement.—The art of manufacturing hydraulic cements seems to have been lost in the Eastern Hemisphere during the Middle Ages, while it also passed away with the decline of the early civilization in the Western Hemisphere. John Smeaton, in 1756, when building Eddystone lighthouse, discovered that argillaceous limestones produced limes that would set under water, and thus rediscovered hydraulic cement. His investigations were carried far enough to secure a good hydraulic lime or natural cement, which, through its durability in the Eddystone lighthouse, secured to Smeaton lasting engineering fame.

Early Manufacture of Natural Cement.—Joseph Parker in 1796 manufactured a species of natural cement, which he called Roman cement, by calcining and crushing septaria nodules found on the Isle of Sheppey, off the coast of Kent, England. Natural cement was also produced at Boulogne, France, in 1802, from septaria, called Boulogne Pebbles.

M. Vicat, during the years 1813-18, produced hydraulic cement by mixing chalks and clays. In the United States, the first natural cement was made in 1818 by Canvass White, from natural rock, near Fayetteville, New York, and was used in the construction of the Erie Canal. Since that time natural cement has been extensively manufactured throughout the United States. During the period from 1818 to 1830, 300,000 barrels were manufactured. The industry gradually increased until the high water mark was reached in the year 1899, when a grand total of 9,868,179 barrels were produced. The production of natural cement has fallen off during the past few years, owing to the reduction in cost of the manufacture of Portland cement, the total output for the year 1905 being only 4,473,049 barrels.

Aspdin Patents Portland Cement.—Portland cement was first produced in 1824, by Joseph Aspdin, a brick mason of Leeds, England, who took out a patent for producing cement by calcining a mixture of lime and clay. He gave it the name "Portland" on account of its resemblance, when hardened, to the famous oölitic limestone used for building, from the quarries on the Island of Portland, in Dorsetshire, on the southern coast of England. The first plant for the manufacture of this cement was established at Wakefield by Aspdin in 1825, and the first important piece of engineering work in which it was used in any quantity was in the construction of the Thames tunnel in 1828. The quality of the cement was greatly improved during the years 1845-50, due largely to the exertion of John Grant, an eminent English engineer, who used it extensively on the London drainage works.

For a time England led in the manufacture of Portland cement, but Germany took the lead in its production, and, until the past four or five years, has been the foremost country in the production and use of Portland cement. During the past few years, however, the United States has surpassed all other countries as a manufacturer and user of Portland cement.

The first American Portland cement was manufactured by David O. Saylor, of Coplay, Pa., in 1875. The development of this new industry was so slow, however, that in 1890 only 335,500 bbls. were manufactured in the United States. Since that time the development of the industry has been rapid, reaching a grand total of 35,246,812 bbls. in 1905, over one half of this being produced in the Lehigh district of Pennsylvania and New Jersey.

Classification and Manufacture of Cement-Concrete Defined.—Concrete is a species of artificial stone formed by mixing cement mortar with broken stone or gravel. Sometimes the broken stone or gravel is replaced by cinders, slag or coke, making a lighter but weaker concrete, especially adapted for fireproof floors. The cement is the active element of the concrete, and is sometimes called the *matrix*, while the sand and broken stone which form the body of the mixture are inert materials and are called the *aggregate*.

Reinforced Concrete Defined.—Reinforced concrete, sometimes called concrete-steel, ferro-concrete, or armored concrete, is a heterogeneous material utilized in construction, and composed of a metal skeleton-work imbedded in a mass of concrete or cement mortar.

Iron, in the form of rods and bars, has been used to tie together and strengthen masonry structures for hundreds of years. Its use, however, was confined to cut stone masonry in the form of clamps and dowel-pins. Cut stone and rubble masonry do not adapt themselves to the use of iron rods, to take care of tensile strains, hence not until after the advent of modern concrete do we find masonry structures having a metal reinforcement.

First Use of Reinforced Concrete.—The first authentic record of the use of reinforced concrete was at the World's Fair in Paris, in 1855. At that time a small row boat, Fig. 1, built by M. Lambot, having a sheet of cement mortar 1½ inches thick, reinforced by a wire netting, was on exhibition. This boat is still in use at Meraval, France.

At a somewhat earlier date a trellis of iron bars was used by a number of builders in the construction of slender cement fireproof partition walls.

In 1865 François Coignet explained the principles of reinforced concrete, and proposed methods of application for the construction of slabs, arches, large pipes, etc.

Fig. 1.—Lambot's Boat of Reinforced Concrete, 1855.

To F. Joseph Monier, sometimes called the father of reinforced concrete, who first took out a patent in 1865, is given the credit for the invention of this new form of construction. His patents related to the combination of iron and cement mortar, in the construction of basins and tubs for use in horticulture. Monier, who was, it is said, a gardener, while constructing some tanks and reservoirs, wished to reduce the thickness of the walls, and conceived the idea of increasing their strength by incorporating within them a metal trellis work. He persisted in his idea, and for a number of years constructed reinforced concrete troughs, pipes, reservoirs, etc., but it is not probable that he even suspected what a marvelous growth his conception would have. Neither Monier nor any of his countrymen appreciated the scientific value of his idea, and it was the Germans who first developed this form of construction.

Early Use in America.—While the French and German engineers were bringing about the development of the Monier system, American inventors seem to have worked out independently the general principle of reinforcing concrete with iron rods to supply the necessary tensile strength in beams and slabs.

Probably the first man to use these materials scientifically, the metal being buried in the lower or tensile side of the concrete, was W. E. Ward, who in 1875 constructed a building at Port Chester, N. Y. In this building, not only the exterior walls, cornices and towers, were formed of concrete, but all the beams, and the roof were made entirely of concrete reinforced with light

Fig. 2.—Reinforced Concrete Arch Bridge built in 1889, at Golden Gate Park, San Francisco, Cal.

iron beams and rods. Ward built rods into the lower sides of his beams and joists, much as they are built to-day. Not having any formulae to guide him, he relied entirely upon his judgment in proportioning them. Fig. 2 shows what was probably the first reinforced concrete bridge built in the United States. It was constructed in 1889, by Ransome & Smith Co., at Golden Gate Park, San Francisco, Cal.

Mr. Thaddeus P. Hyatt, a native of New Jersey, but at the time living in London, while studying the question of fireproof floor construction, conceived the idea of making beams of concrete, strengthened by imbedding iron bars in their lower edges to care for the tensile stresses. He made many experimental beams, introducing the iron rods in a great variety of ways and employed Dr. David Kirkaldy, of London, to make a series of tests on reinforced concrete beams. The results of these tests were published by Mr. Hyatt in 1877. Unfortunately the edition was limited and the book has long been out of print. These researches were of great value in the development of the science.

In 1877 Mr. H. P. Jackson, C. E., of San Francisco, applied Mr. Hyatt's invention to building construction, and from that date forward used the new form of construction whenever possible.

To Mr. Edwin Thacher is largely due credit for the successful introduction of reinforced concrete bridges in the United States.

Later Developments in Europe and America.—The development of the Monier system dates from 1880, when the patents of this inventor for Austria-Hungary were secured by a German company. Under the management of G. A. Wyss, experiments were made, and principles to be followed in its application were established. Gradually this form of construction came into popular favor throughout the German Empire, and it may truthfully be said that to the Germans is largely due the successful development of reinforced concrete.

During the years 1889 to 1894 a new impetus was given to this method of construction by the inventions of M. Bordenave, Cottancin, F. Hennebique, Edmund Coignet in France, Möller, Rabitz, Könen in Germany, Wünsch in Hungary, Melan in Austria, and Ransome in the United States. Some ten or twelve years ago F. Von Emperger introduced the Melan system in the United States, and constructed a number of Melan arch bridges. Since that time hundreds of arch bridges have been constructed after this system.

CHAPTER I.

CLASSIFICATION AND MANUFACTURE OF CEMENT.

Cement may be defined as a pulverized material, composed principally of silica, alumina and lime, which, when mixed with water, undergoes a chemical change forming new compounds that develop the property of setting or crystallizing into a solid mass even under water.

Classification of Cements.—Cementing materials naturally fall into two groups—*non-hydraulic* cements and *hydraulic* cements.

Non-hydraulic cements are made by burning either gypsum or pure limestone at comparatively low temperatures. The products obtained by burning gypsum are known as plaster of Paris, Keene's cement, cement plaster, etc. The product of burning limestone is common lime. While limes and plasters are extensively used for building purposes, they are not used in reinforced concrete construction.

Hydraulic cements are those which set under water, and are included under the following four general classes:

1. Hydraulic Limes.
2. Natural Cements.
3. Portland Cements.
4. Puzzuolana Cements.

Hydraulic Limes.—Hydraulic limes have been defined as the products obtained by the burning of argillaceous or silicious limestones, which, when showered with water, slake completely or partially without sensibly increasing in volume. Argillaceous limestones used in the manufacture of hydraulic limes usually contain from 10 to 20 per cent. of clay homogeneously mixed with carbonate of lime as the principal ingredient. Silicious limestones contain from 12 to 18 per cent. of silica, small percentages of oxide of iron, carbonate of magnesia, etc.

No hydraulic limes are manufactured in the United States, and, while they are manufactured extensively in certain localities in Europe, the subject is not of sufficient interest to warrant a description in this place of the methods of manufacture.

Natural Cement and Its Manufacture.—Natural cement is the product resulting from the burning and subsequent pulverization of a natural clayey limestone (containing 15 to 40 per cent. of silica, alumina and iron oxide), without preliminary mixing and grinding, the heat of burning being insufficient to cause vitrification. During the burning the carbon dioxide of the limestone is almost entirely driven off, and the lime combines with the silica, alumina, and iron oxide, forming a mass containing silicates, aluminates and ferrites of lime; or, if magnesium carbonate is present in the original rock, magnesium compounds will result. It is necessary to grind this burned mass rather fine, for it will not slake as it comes from the kiln if water be poured on it. This finely ground powder when mixed with water, hardens or sets rapidly, either in air or in water.

American natural cement was formerly called Rosendale cement, due to the fact that it was first manufactured at Rosendale, N. Y.

The manufacture of natural cement from a mechanical standpoint is a comparatively simple process, consisting of burning the rock as it comes from the quarry, in plain upright kilns, and grinding the burnt friable pieces to a powder. The rock in its natural state contains the proper ingredients for natural cement. The limestone is usually stratified, the strata varying somewhat in chemical composition. Several strata are usually mixed for any given brand of cement, the idea being that if one layer contains too much silica it may be corrected by another containing too much lime or magnesia. The rock is either quarried in open cut where the stripping is light, or is mined by means of tunnels and chambers. The rock, as it is quarried, is broken into sizes convenient for handling, and then run through an ordinary rock crusher, which breaks it into pieces varying in size up to six inches; then it is conveyed, usually by an ordinary tramway, directly to the loading platform at the top of the kiln.

With but few exceptions, the kilns used in the American natural cement industry are of the vertical continuous mixed-feed type. They are commonly about 45 ft. high and 16 ft. in diameter, and are built of masonry, lined with firebrick, or have an iron shell, lined with fire brick. Fig. 3 shows a vertical section of such a kiln. The rock and fuel are spread in the kiln in alternate layers, the proportion of fuel being regulated by the man in charge of burning. Either anthracite or a good quality of bituminous coal is used, according to the locality. When anthracite coal is used it requires about 10 lbs. of coal to burn 100 lbs. of rock. The temperature of burning varies according to the character of the rock. It is somewhat greater than that used for burning lime, but is generally considerably below the point of incipient fusion reached in burning Portland cement.

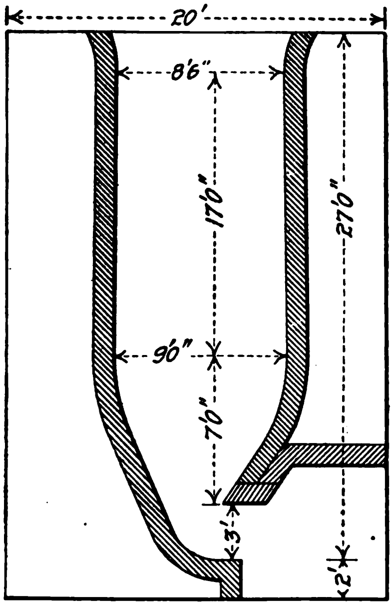

Fig. 3.—Kiln for Burning Natural Cement.

It is impossible to burn the rock uniformly, hence it is necessary to sort out and throw away the under burnt and over burnt clinker. Bad weather, bad management, the character of kiln used, etc., determine the amount of loss, which varies from 10 per cent. under the best conditions to 33 1-3 per cent. under bad conditions, with a probable average loss of about 25 per cent. The sorted calcined rock is conveyed to crushing machines, usually of the rotary type, such as the "pot cracker," consisting of a ribbed, steel-faced, or chilled iron, cone revolving within a corrugated conical shell, as shown in Fig. 4.

The material is conveyed from this machine to screens, which take out the cement that is fine enough to pack. The coarser particles go to fine grinding machines. These machines may be either edge runners, ball or tube mills, or ordinary mills, or emery faced stones. The methods used during this part of the process are essentially similar to those employed in the manufacture of Portland cement. The product passes from the reducing mills to the mixers by means of which a more thoroughly uniform

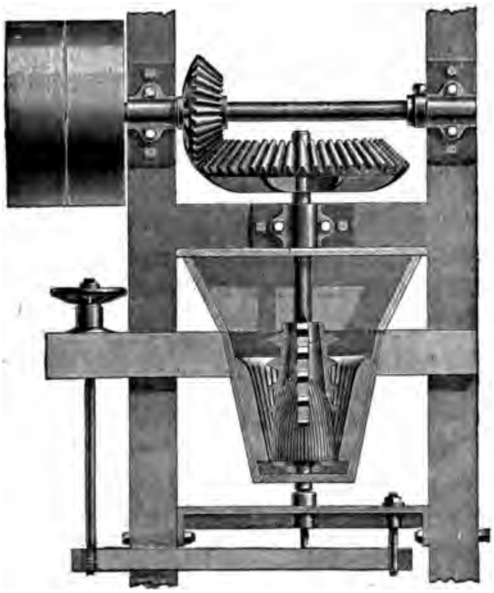

Fig. 4.—Pot Cracker for Natural Cement Rock.

product is obtained. It is then conveyed by chutes to the bags and barrels in which it is packed.

The cost of manufacture varies with local conditions. The various items which go to make up the cost are: cost of quarrying, or mining the rock, cost of labor at the kilns and mill, cost of fuel for the kiln, cost of power, interest and depreciation of plant. These may vary from 15 to 50 cts. per barrel of cement manufactured.

Portland Cement and Its Manufacture.—Portland cement is an artificial product obtained by finely pulverizing the clinker produced by calcining to incipient fusion a natural or artificial mixture of finely ground argillaceous and calcareous materials, this mixture consisting approximately of three parts of carbonate of lime or lime oxide to one part of silica, alumina and iron oxide. The essential components of Portland cement are silica, alumina and lime; while the ingredients always occurring with these in appreciable quantities are iron, magnesia, alkalies, sulphuric and carbonic acids, and water. These ingredients should approximate the following limits given by Le Chatelier for commercial Portland cement:

	Per cent.
Silica	21 to 24
Alumina	6 to 8
Iron Oxide	2 to 4
Lime	60 to 65
Magnesia	0.5 to 2
Sulphuric Acid	0.5 to 1.5
Carbonic Acid and Water	1 to 3

The materials from which Portland cement is manufactured vary with the locality, and usually consist of either cement rock and limestone, limestone and clay, marl and clay, chalk and clay or slag and limestone. Cement rock and limestone are chiefly used in the Lehigh district, and constitute the raw materials used for two-thirds of the Portland cement manufactured in this country. Limestone and clay are the materials used in the New York State cement region, marl and clay are used in the cement mills of the middle west. Chalk and clay are the materials used in the states bordering the Mississippi River on the west and in Texas. Slag and limestone, although extensively used for the manufacture of cement in Europe, have as yet been little used in this country.

For a more extended discussion on the raw material used for the manufacture of Portland cement, see "Cements, Limes and Plasters," by Edwin C. Eckel.

In the early days of the industry, Portland cement was calcined in stationary kilns similar to those used in the manufacture of natural cements. This type of kiln is still occasionally used in this country, and is used to a larger extent in France and Germany. Although the coal consumption is smaller than with the rotary kiln, labor is a much larger item, and on this account the stationary kiln is not an economic method of manufacture unless the cost of labor is quite low. The only essential difference between this method and that used for the manufacture of natural cement consists in the grinding and mixing of the raw material while wet, and moulding the mix into bricks, which are dried before being calcined.

The Dry Process of Manufacture.—Rotary kilns are used almost exclusively for the manufacture of cement in the United States. Two processes are employed, the dry process and the wet process. The dry process with rotary kilns may be considered as the

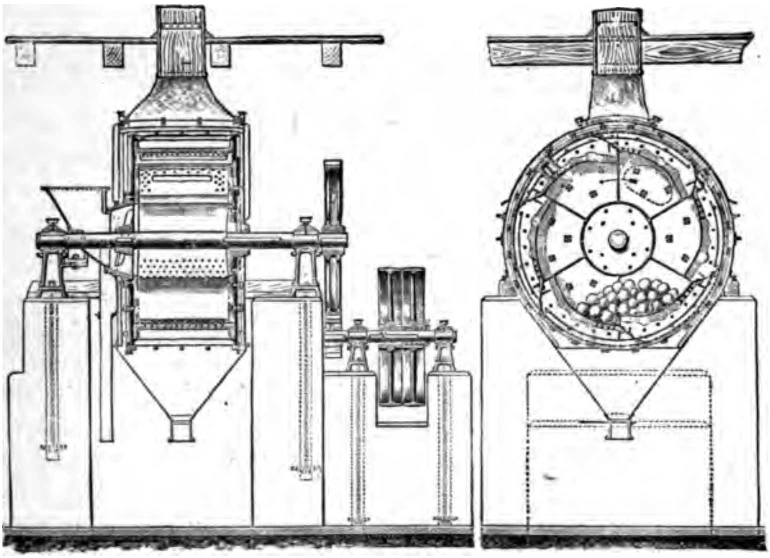

Fig. 5.—Ball Mill of the Krupp Type.

typical American method for the manufacture of Portland cement. The wet process does not differ materially from the dry process.

The dry process is adaptable to any class of materials, which can be quarried and pulverized in a dry state, and is briefly as follows: The raw material is conveyed from the quarry to the mill and is first passed through crushers, which reduce it to a maximum diameter of 2 or 3 ins. It is then conveyed to storage bins, where it remains until the chemical composition has been determined, so that the mix can be properly proportioned. A suitable mixture by weight is then made and conveyed to a dryer, which is kept at a temperature sufficiently high to drive off the greater part of the moisture contained in the rock. The dryer usually consists of a rotary cylinder 4 or 5 ft. in diameter, 40 to 50 ft. long, with its axis slightly inclined to the horizontal.

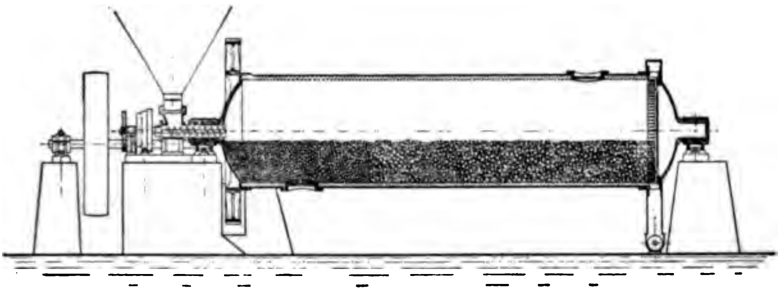

Fig. 6.—Tube Mill of the Davidsen Type.

The materials enter at the upper end and are discharged at the lower end. Heat is usually supplied by a small furnace.

From the dryer, the material is conveyed to a preliminary grinding machine, usually of the ball mill type, which reduces it to a size small enough to pass a No. 20 or No. 30 mesh sieve.

Fig. 7.—Griffin Mill.

Figure 5 shows the usual type of ball mill. The mixture then passes to the fine grinder, where it is further reduced until from 90 to 95 per cent. will pass a No. 100 sieve. The tube mill (Fig. 6) or Griffin mill (Fig. 7) is usually employed for fine grinding.

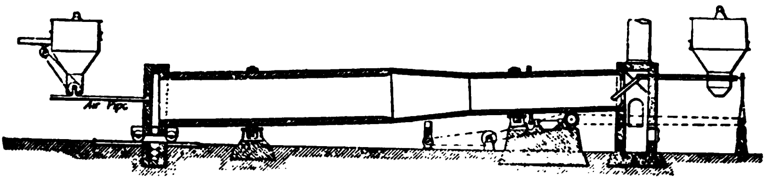

Fig. 8.—Longitudinal Section of Rotary Kiln.

From the grinding machines the mixture is conveyed to bins above the rotary kilns into which it is fed automatically.

The rotary kiln (Figs. 8 and 9) is a steel cylinder, varying in length from 40 to 150 ft. and from 4½ to 9 ft. in diameter, lined with from 6 to 12 ins. of fire brick, with its axis inclined 8 or 10 degrees to the horizontal, and arranged to rotate at a speed averaging about one turn per minute. The raw materials are introduced at the upper end in the form of powder, and in passing through are calcined to a clinker, which leaves the kiln at the lower end in small balls, ranging from ⅛ to 1½ ins. in diameter. Finely pulverized gas slack coal is generally used for fuel, although both gas and oil have been employed, but with poorer results. The coal is blown into the lower end of the kiln, and instantly ignites, forming a flame reaching from 15 to 25 ft. into the kiln, and producing a temperature of from 2,600 to 3,000 degrees Fahrenheit. The coal is pulverized in the same manner, and to about the same degree of fineness as the raw materials. The temperature and time of burning vary with the nature of the raw materials.

The clinker as it leaves the kiln is sprayed with a small stream of water, which cools and makes it more easy to pulverize. It then passes through coolers, which reduce it to a normal temperature. From the coolers the clinker passes to the pulverizing and grinding machines, which are similar to those used for re-

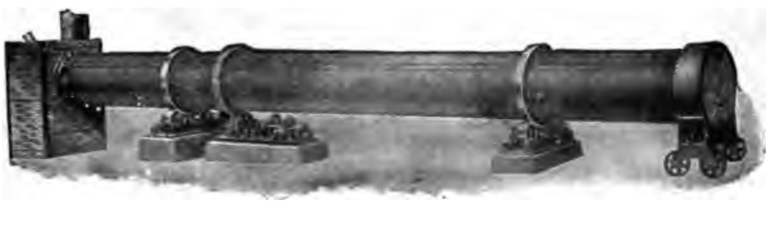

Fig. 9.—Rotary Kiln as Made by the Bonnot Co.

ducing the raw material. The finished cement from the grinding machines is conveyed to the stock house, often being stored for a time to give it a chance to "season" somewhat. It is then packed in bags or barrels for shipment.

The Wet Process.—The wet process may be used either with rotary or stationary kilns. In the United States it is usually only employed by the mills in which the raw material used is marl, although it is adapted to chalk or other materials, which are easily reduced when in a wet condition. When water is used in reducing the material, less power is needed for operating the machinery. The saving in this part of the process is, however, more than balanced by the cost of the additional coal needed to evaporate the water from the slurry after it is fed into the kiln. The cost of handling wet material is less than dry material, as it may be pumped from one part of the plant to another.

Wet Process With Rotary Kilns.—If we assume that marl and clay are used, the process is as follows: The marl, after excavation, is passed through a disintegrator and sometimes a stone and grass separator, and run into storage basins, while the clay is dried, pulverized, and then mixed with a proper amount of marl in pans of the edge runner type (Fig. 10) the slurry containing enough water to give it a thick creamy consistency. In some mills this process is varied by mixing the clay with water before adding it to the marl. The mixture is then ground, while still in a wet condition, in either edge runners, or tube mills, from which it is run into slurry tanks, where it is kept agitated by revolving paddles or by compressed air, and from which chemical analyses are made to check the accuracy of the proportions, corrections being made if necessary. Centrifugal pumps and compressed air are both used for handling the slurry.

The wet slurry is then pumped directly into the upper ends of rotary kilns, which are usually somewhat longer than those employed in the dry process, so that the waste heat may be utilized in driving off the excess water. About 150 to 160 lbs. of coal per barrel of cement are necessary for the burning, which is from 30 to 50 per cent. more than required in the dry process, but this disadvantage is largely compensated by the cheaper method of handling and preparing the raw material. The treatment of the clinker is similar to that of the other processes.

Wet Process With Stationary Kilns.—In this process the clay and marl, or chalk, are first ground, if necessary, and then mixed together in a wash mill with a large excess of water, the lumps being broken up by means of agitators. When the materials have thus been reduced to a very finely divided state, the mixture is run into settling basins, where the solid matter settles and

Fig. 10.—Mixing Pan for Marl and Clay. Edge Runner Type.

from which the excess of water is drawn off. The slurry, when still further hardened, is formed into bricks and burned in stationary kilns.

A modification of this method, known as the semi-wet process, consists in mixing with a smaller amount of water, sufficient to give a creamy consistency, the operation being similar to the wet process with rotary kilns, except that the slurry is partly dried and formed into bricks instead of being fed directly into the kilns. The chief disadvantages of the process are the large space necessary for settling and drying the slurry and the greater amount of labor required. It is used extensively in Europe, and in England a few years ago might have been considered the typical process. It is, however, not used in this country.

Portland Cement from Blast Furnace Slag.—This has been manufactured in Europe for several years, but its manufacture has only recently been undertaken in the United States. As the method involves the utilization of the waste products from the blast furnace, it is likely that it will become popular., The method of manufacture is briefly as follows:

The slag, as it comes from the blast furnace, is sprayed with water, which granulates it and changes its chemical composition, the water combining with the calcium sulphide, which is injurious to cement, to form a lime and sulphuretted hydrogen. The granulated slag is then dried, mixed with the correct proportion of dried limestone, and ground to extreme fineness. The mixture is then burned in a rotary kiln. The remainder of the process is essentially the same as that already described for the manufacture of Portland cement from ordinary material.

Slag Cement and Its Manufacture.—Slag or puzzuolana cement is made by intimately mixing granulated blast furnace slag of proper composition with slaked lime, and reducing this mixture to a fine powder. This product differs materially from Portland cement, although it is sometimes called a Portland cement by the manufacturers. While it is an excellent material for many purposes, it possesses certain qualities which prevent its use as a substitute for Portland cement in many classes of work. The largest piece of work in the United States, known to the author, upon which slag cement has been used to any extent, was the drainage system for New Orleans.

The method of manufacture is briefly as follows: Slag of the proper composition is chilled as it comes from the furnaces by the action of a large stream of cold water under high pressure. The slag is, thereby, broken up; about one third of its sulphur is eliminated and it undergoes other chemical changes. A sample of the slag thus granulated is mixed with a proportion of prepared lime, and ground in a small mill, thereby producing a small amount of actual slag cement. If the tests upon this trial cement are satisfactory, the slag is dried and then ground, first in a Griffin mill and then in a tube mill. Then it is mixed with the proper amount of prepared lime, and the two materials are ground and intimately mixed together. The resulting product is said to be so fine that 95 per cent. will pass a No. 200 sieve.

The lime is burned from a very pure limestone, and stored in bins, beneath which are two screens of different mesh, the coarser at the top. A quantity of lime being drawn on the upper screen, is slacked by the addition of water containing a small percentage of caustic soda. The lime passes through the two screens as it slakes, and is then heated in a drier, the slaking being thus completed. The lime may then be incorporated with the slag. The purpose of the caustic soda used in the above process is to render the cement quick setting.

CHAPTER II.

PROPERTIES OF CEMENT AND METHODS OF TESTING.

Portland cement is used for reinforced concrete construction, almost to the exclusion of other cements. Its great strength, uniform composition and the regularity of its properties eminently fit it for this class of work. In manufacture, the distinguishing characteristics of Portland cement are the use of an artificial mixture, the grinding of the raw materials before burning, and the calcining to incipient fusion. In use, its distinguishing characteristics are its high specific gravity, dark color, slowness of setting and great strength.

Natural, quick setting cements are used for reinforced concrete only in special forms of construction, viz., in repair work, as when quick setting is necessary in order to enable the structure to sustain moderate loads or enable its use within a few hours; in hydraulic work, as in the construction of reservoirs and conduits; and in the construction of reinforced concrete pipe. They are, however, extensively used for plain concrete work. Sometimes when quick setting with great strength is desired, a mixture of natural and Portland cement is employed. In manufacture, the distinguishing characteristics of natural cement are its production from a single variety of material, unground and burned at a low temperature; and in use, its lighter weight and color, quick setting property, and small strength in the early stages of hardening.

Slag cements, as yet, have not had extensive use in this country. They are characterized, in manufacture, by their production from intimately mixed granulated blast furnace slag and slaked lime, without the usual process of calcining employed in the manufacture of other cements. In use, slag cements are commonly distinguished by their light color, inferior specific gravity, slow set and lower strength. The low strength, variable composition and uncertain properties, of both natural and slag cements, render them undesirable for reinforced concrete structures.

Field Inspection.—Cement is usually sold in barrels, or in cloth or paper bags. When in danger of being subjected to dampness in shipping from the place of manufacture to the site of the work, barrels are employed; but in the majority of cases, cement is shipped in cloth bags. Cement is generally stored temporarily at the site of construction on raised platforms for about 10 days, in order that the necessary tests may be made. At the time of delivery the condition of the packages should be observed: they should be plainly marked with the brand and name of the manufacturer. A field inspection often enables a correct judgment to be formed of the condition of the cement. Old or well-seasoned cement is generally lumpy, but the lumps are easily broken up. If, however, the cement has been subjected to excessive dampness, or has been wet, lumps will be formed which are hard and difficult to crush. This cement is probably hydrated and of inferior quality. It should also be noted whether the cement runs uniformly in color, as a change in color may indicate a change in brand or quality, and should lead to careful testing.

Sampling.—For the purposes of testing, samples should be taken from bags at random. There are several good methods of sampling, but perhaps the most satisfactory is to take a small sample from each of a number of bags, mix these lots together and separate the same into a convenient size for testing. When a sample from a single bag is taken, it is usually stipulated that a sample shall be taken from one bag in ten, the bag being picked out at random. When small lots of cement are used, the samples should be taken more frequently, a sample from every five bags being about right. Care should be taken that the sample be representative of the material in the bag, part being taken from the surface and part from the interior. Usually a sample weighing 8 to 10 lbs. will be enough for the ordinary purpose of testing. Samples should be placed in a tightly covered can and stored in a dry place until tested.

Properties of Cement.—In order that cement shall come up to the requirements necessary for a high class of work, it must possess certain desirable properties, and be free from others which may be injurious. The desirable elements are: (1) That when treated in the proposed manner it shall at the end of a definite period develop a certain strength; (2) that it shall contain no compounds which may at any future time cause it to change its form or volume, or lose any of its strength; and (3) that it shall withstand the action of any outside agency which may tend to decrease its strength or destroy its durability. When a cement fulfills these requirements it will be a safe and satisfactory construction material. To determine whether it fulfills these requirements, certain properties must be considered and certain tests made to determine other properties. In determining the value of a given cement for structural purposes, the qualities usually considered are: (1) Its color; (2) specific gravity; (3) activity; (4) soundness; (5) fineness; (6) strength, and (7) chemical composition.

In the examination of a given sample of cement its failure to conform to the usual requirements in regard to any one of these qualities should not necessarily lead to its condemnation, but rather classify it as suspicious, and it should be tested carefully in every possible manner before accepting or rejecting.

Color.—While the color of cement has little bearing upon its quality, it may indicate an excess of some one ingredient; and for any given brand, variation in shade may indicate differences in the character of rock used, or in the degree of burning.

Portland cement should be a dull gray. Bluish-gray probably indicates an excess of lime; dark green, a high percentage of iron; brown, an excess of clay; and a yellowish shade indicates over burning.

Natural cements vary greatly in color, ranging from a light yellow to dark gray, and even to a chocolate brown. Generally the color is no criterion of quality, but may be considered as giving some indication of the uniformity of a given grade or brand of cement.

Slag cements are usually much lighter in color than Portlands, and slightly different in tint, while they differ markedly in tint from most natural cements. They are commonly bluish-white to lilac, the exact color of any specimen depending partly on the respective colors of the lime and slag which have been used in its manufacture, but more largely on the relative proportions in which these two ingredients have been mixed. Slag cement, after being kept under water, shows, when fractured, a bluish-green tint, which is supposed to be due to the presence of sulphide of calcium. Slag cements do not stain masonry, hence they will have an extended use in architecture.

Specific Gravity.—The specific gravity of a substance is the ratio of its weight to the weight of an equal volume of water. As the specific gravity of a well-burned cement is known to have certain definite limits, the specific gravity of a cement may be said to give a true indication of the thoroughness of burning. The higher the temperature used in burning, the more thoroughly will the ingredients be combined; and it follows that their volume will contract, resulting in a greater density or higher specific gravity. Too high a specific gravity will therefore indicate over burning. Over burning tends to break up some of the compounds which should be present in a normal cement, and to form others that may not be injurious, but, nevertheless, possess such feeble hydraulic properties that they tend to weaken the material.

A low specific gravity indicates under burning, adulteration and hydration. An under burnt cement contains a large proportion of uncombined, or insufficiently combined, elements, some of which are sources of great danger. If the cement is used, these elements may cause disintegration and the ultimate failure of the structure.

Adulteration, which may be detected by the specific gravity test (excepting adulteration with gypsum or plaster of Paris, of which there is a legitimate use), may consist of the incorporation of raw-rock, cinder, slag and natural cement. All these ingredients have a lower specific gravity than Portland cement. If the Portland cement is of high grade, as high as 20 or 25 per cent. of adulterants may at times be added, and the cement will still possess sufficient strength to pass the usual physical tests. The incorporation of so high a percentage of impurities, which possess a much lower specific gravity, is at once apparent when the specific gravity test is applied to the mixture.

The specific gravity test, however, should not be relied upon alone for the detection of adulterants, since many other causes may operate to produce an abnormably low value, as the age of the cement, fineness of grinding, composition of material, etc. It should be taken as indicative, and should be verified by other tests before rejecting a material which does not come up to standard.

The specific gravity of natural cement is generally no criterion of its quality, but, to some degree, may be regarded as a measure of the uniformity of a single grade.

The specific gravity of Portland cement varies from 3.00 to 3.25, but for the higher grades of American cements it is usually found to be between 3.10 and 3.25. The specific gravity of natural cement varies from 2.75 to 3.05. Slag cements are lighter than Portland, and in some cases lighter than natural cements, their specific gravity usually ranging from 2.6 to 2.9. A slight variation in the specific gravity often denotes a considerable difference in the quality of a cement, hence great care is necessary in making this test.

The use of Le Chatelier's * apparatus, is recognized as the standard method of determining the specific gravity in American practice, and is recommended by the committee on Uniform Tests of Cement, of the American Society of Civil Engineers.

Activity.—When cement is mixed into a paste with water and allowed to stand, it gradually hardens. The rate of hardening is termed the time of setting or activity. Two distinct stages in setting are recognized: (1) the initial set; and (2) the hard set. The first takes place when the mass begins to harden; and the second, when the hardening has reached a point where the mass can not be appreciably distorted without rupture. The determination of the first period is important, as the material must be deposited and remain undisturbed before the point is reached, for otherwise a great loss of strength will result.

The time of setting may vary within wide limits, and is no criterion of the quality of cement. However, a cement may set so quickly that it is worthless as a construction material, or it may set so slowly that it will greatly delay the progress of the work. Again, after it has been placed in the structure it should set and harden as quickly as possible, so that it can offer resistance to any external forces. Hence certain definite limits must be fixed for the time of setting. The best cements should be slow in acquiring initial set; but, after having reached that point, should harden quickly.

A natural cement is generally much quicker in setting than

*For an exhaustive description of the apparatus and method used for this determination see "Practical Cement Testing," by W. Purves Taylor, New York, 1906.

a Portland, although slow setting natural cements are occasionally met with. In natural cements the hard set frequently occurs within a few moments after the initial set, sometimes within a period of 15 minutes, and should develop hard set in from 30 minutes to 3 hours. Initial set should in no case develop in less than 10 minutes.

Portland cement should develop initial set in not less than 30 minutes, and hard set in not less than one hour, nor in more than 10 hours.

Normal slag cements are slower setting than Portland. At times burnt clay, or slags high in alumina, or certain active forms of silica, etc., are added to increase the activity, the attempt being made to obtain a cement with about the same activity as that possessed by Portlands.

The composition, degree of burning, age, fineness of grinding, amount of water used in mixing, and the temperature and humidity of the air, affect the activity of a cement. It is usual to add a small percentage of gypsum or plaster of Paris during manufacture, to retard the setting. Small percentages of these materials increase the strength of cement, but larger quantities may cause it to blow or expand. A greater quantity than 2 per cent. is dangerous. A lightly burned cement, or a freshly burned cement, sets more rapidly than a hard burned or an old cement. This is due to the presence of non-hydrated free lime. Hence care should be taken during manufacture to secure sufficient burning.

A moderate amount of "seasoning" is also helpful to secure good results in use; for, if cement is allowed to stand exposed to the air and to dampness, it gradually absorbs water and carbonic acid. These produce a chemical change in the materials, resulting gradually in slower setting, and eventually the cement loses all its hydraulic properties, although a well protected cement may be stored for a long time without appreciable deterioration. Aging, therefore, under usual conditions is not to be desired. The effect of age on setting is generally less noticeable with natural than with Portland cements.

The activity of a cement varies somewhat with the amount of water used in gauging; the greater the amount of water used the slower the setting. The temperature of the water also affects the setting; high temperatures accelerate the setting. A finely ground cement is almost invariably quick setting, unless artificially retarded. This is due to the fact that a finely ground material is more quickly attacked by a solvent than a coarser one. The temperature and amount of moisture in the air also affect the activity of a cement, high temperatures and a dry atmosphere increasing the activity, while low temperature and a humid atmosphere retard the setting.

For the apparatus and method used to determine the time of set consult "Practical Cement Testing," by W. Purves Taylor, New York, 1906.

Soundness.—The soundness of a cement refers to the property of not expanding, contracting or checking in setting. It is absolutely necessary that the cement shall neither shrink nor swell after the process of setting has once begun. When the ingredients of Portland cement have been improperly mixed, or the process of manufacture has been improperly carried on, the cement will have a tendency to expand, crack and disintegrate after the setting has commenced. Unsoundness is generally due to an excess of lime, either free or loosely combined, which has not had an opportunity to become sufficiently hydrated. The presence of this lime may be due to incorrect proportioning, to insufficient grinding of the raw material, to under burning, to lack of seasoning or to insufficient grinding of the calcined rock. The presence of sulphides, an excess of magnesia or of the alkalies, may also cause expansion and disintegration, and at times may be more harmful than uncombined lime. Contraction is sometimes due to an excess of clay.

The age of a cement greatly affects its soundness. Almost every cement, no matter how well proportioned and burned, contains a small amount of free or loosely combined lime, which may cause unsoundness if the cement is tested before attaining sufficient age. This lime, however, if exposed to the air, will hydrate in a short time, becoming inert. In many cases, when a fresh cement tests unsound, it will be found that if it is stored for two or three weeks this unsoundness will disappear. Fineness of grinding is essential to perfect hydration, and it will be found in most cases that a coarsely ground cement is an unsound one, the larger particles not being readily subjected to hydration.

Tests for soundness are among the most important to be made upon cements, and should extend over considerable time to fully develop possible inherent defects. The usual manner of determining whether or not a cement is sound, is to immerse in water a small pat of neat cement mortar, 2 or 3 ins. in diameter, with thin edges. This pat is examined from day to day to see whether it cracks or in any way becomes distorted. Another pat is allowed to set in air, and is examined for blotches and discoloration.

Another test for soundness is by measuring the amount of change in volume. A rough method is to press some mortar firmly in a glass tube or lamp chimney. If a dangerous amount of expansion takes place the glass will be broken. Shrinkage may also be determined by pouring some colored liquid into the chimney after the cement has thoroughly set. An idea of the amount of shrinkage may be formed by the amount of liquid that runs down the inside. Several more accurate, but much more complicated methods for making this test, are used in larger testing laboratories. "Practical Cement Testing," by W. Purves Taylor, should be consulted if an elaborate discussion on this subject is desired.

Accelerated tests are widely used and are designed so to hasten the action of the expansive ingredients that the same results will be produced within a few hours, or at most a week, that under normal conditions will not appear for weeks, months, or even years. These tests consist of placing a pat made of neat cement of normal consistency, and usually moulded upon glass, either in warm, hot or boiling water, or in steam for several hours. These severe conditions tend to warp or disintegrate unsound cements. In the Faija test, warm water at a temperature of 115° F. is used. In what is known as the "Hot Water" test a temperature ranging from 130 to 200° F. is maintained.

In the boiling test the specimens are subjected to the action of boiling water from one to six hours. Sometimes the pat is subjected to an atmosphere of steam above boiling water for 3 hours, or, when 24 hours old, is subjected to a steam bath for 3 hours, and then is boiled for from 2 to 6 hours. These tests are all more or less satisfactory, depending upon the degree with which they corroborate other tests for soundness. Taylor states "that of a large number of tests which failed in the boiling test, 86 per cent. gave evidence in less than a year of possessing some injurious quality, and that, of those cements that passed the boiling test, but one-half of one per cent. gave signs of failure in the normal test pats, and but 13 per cent. showed a falling off in strength in a year's time." On the other hand, while conceding the value of this test, it often happens that a cement may pass the boiling test well, yet check and disintegrate in the normal tests. Again, cements have passed sound which would not pass the boiling test. Hence we should consider this test as a corroborative test only, and not as final. Lastly, it is safe to assume that if a cement passes the boiling test it may be considered safe until the results of the normal tests are known, and, if it does not pass the boiling test, it should be regarded with suspicion until the results of other tests are available.

For natural cements, tests made on pats of neat cement paste kept in air and water under normal conditions are considered to be the only conclusive ones. Excessive expansion, checking or disintegration on normal pats exhibit similar phenomena in both natural and Portland cements. Accelerated tests have not proved successful for natural cements.

In slag cements, unsoundness is usually due either to unslaked lime or an excess of sulphides or magnesia. If the lime is not thoroughly slaked, or is coarsely ground, it will tend to produce swelling and disintegration as with Portland cement. The effect of sulphur in the form of sulphides is noticeable chiefly in air, where they oxidize to sulphates with a great change in volume, thus causing disintegration. In water this change does not take place, although the pats generally show blotches of bluish or greenish-gray, probably due to the formation of iron sulphides. Tests of slag cement are usually made on normal pats and on specimens submitted to boiling, and, for normal tests, should give no indication of unsoundness, other than blotching at the end of 28 days, and should pass the boiling test. If failure takes place, either test should be sufficient cause for the rejection of the cement.

Fineness.—The finer a cement is ground, the better its quality. Water acts only on the finer particles, while the coarser particles are almost always inert. The finer a cement is ground the greater will be its covering capacity, therefore, the greater its value as a cementing material. To produce the greatest strength each particle of the aggregate should be covered with cementing material. The greatest economy, other things being equal, will result when the cement is as fine as possible. However, while fine cement is more valuable than coarse, fine grinding increases the cost of manufacture, hence there is a limit to the amount of grinding which can be done economically. Again, a finely ground cement is less apt to blow or disintegrate than a coarse one, since the free or loosely combined lime being in fine particles, is thoroughly broken up and readily rendered innocuous by the water when it is added.

A Portland cement of good quality should be fine enough to pass at least 92 per cent. by weight through a No. 100 sieve, and 75 per cent. through a No. 200 sieve. A No. 100 sieve has from 96 to 100 meshes per lineal inch, and a No. 200 sieve from 188 to 200 meshes per inch.

The degree of fineness to which a natural cement is ground depends both upon the composition of the material and the process of grinding used. At times the percentage which will pass a No. 200 sieve will approximate that for Portland cement. Fine grinding is, however, not as essential in the manufacture of natural as in Portland cement, as the amount of free lime present is much less. If the requirements are such that 85 per cent. or more must pass a No. 100 sieve, and 70 per cent. or more must pass a No. 200 sieve, a good quality of natural cement should result.

Slag cements of necessity must be ground much finer than is necessary for Portland cement. It is common practice to require not less than 97 per cent. to pass a No. 100 sieve and from 90 to 92 per cent. to pass a No. 200 sieve.

Strength Tests.—The strength of a cement mortar may be determined by testing it. The object of the test is to obtain a measure of the strength of a material as used in actual work. As a rule tensile tests only are made, although cement mixtures are used almost entirely in compression, and may be subjected to every conceivable form of stress. The reason for this is that the tensile strength is more easily determined, and is more or less a true measure of the compressive, transverse, adhesive and shearing values. Again, investigation appears to show that the strength of cement in tension is more susceptible to any good or bad influences affecting the material, and, therefore, furnishes a better criterion of its value than tests made in any other manner.

There exists a certain definite relation between the tensile and all other strengths, hence the results of the tensile tests give a reliable basis for computing the values of the strength under other forms of stress.

Tests are usually made on both neat cement and sand mixtures.

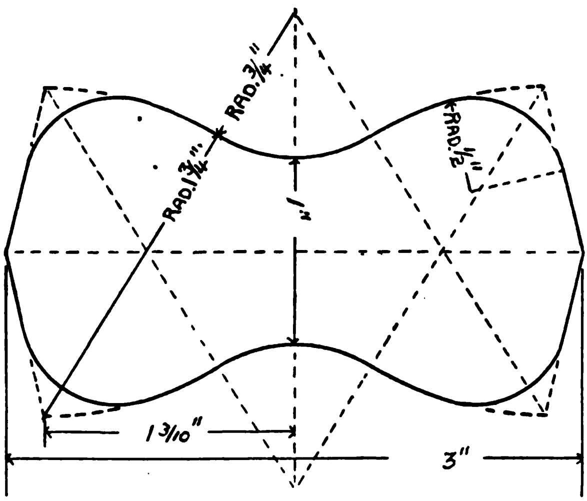

Fig. 11.—Standard Cement Briquette.

While there is no definite relation between the strength of neat and sand briquettes, neat briquettes are more susceptible to both internal and external influences, and are, therefore, better criterions of the character of the material and may be considered as a measure of its quality, while the sand tests are a true measure of the strength under actual conditions.

For sand test of Portland cements, the mixtures are composed of 1 part by weight of cement to 3 parts of sand; while for natural and slag cements, richer mixtures are used on account of the greater weakness of these materials in the early stages of setting, 1 to 1 and 1 to 2 mixtures being employed. Both standard and normal sands are used for these tests. The periods at which the briquettes are broken are 24 hours, 7 days, and 28 days for neat tests; 7 days, and 28 days for sand tests, although for experimental purposes much longer periods are necessary to secure reliable data.

Standard Briquettes.—Tests are usually made with briquettes of standard form, having a minimum cross-sectional area of one square inch. The standard American form of briquettes is shown in Fig. 11. This is the form adopted by the Committee of the American Society of Civil Engineers.

Moulds for briquettes should be made of brass. They are either single, or in gangs of three or four. A simple form is shown in Fig. 12, which is the mould adopted by the Committee of the American Society of Civil Engineers.

Normal Consistency of Mortar.—The amount of water necessary to make the strongest mortars varies with each cement, and is

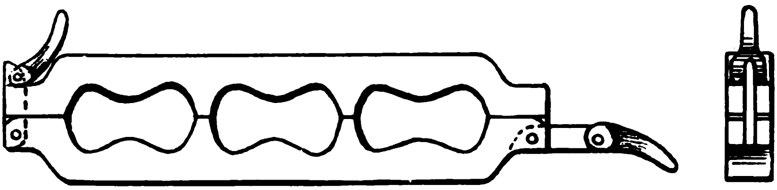

Fig. 12.—Gang Mould for Standard Cement Briquettes.

usually expressed in percentages by weight. No fixed percentage can be adopted, hence it is customary to mix with varying amounts of water until a certain normal consistency of mortar is secured. The amount of water necessary to bring different cements to the same consistency varies with the composition, age, fineness, etc., so that the amount of water must be determined experimentally in each case. A normal consistency determined by what is called the "ball method," is secured in the following manner: Cement paste is mixed to such a degree of plasticity that when a ball of the paste, about 2 ins. in diameter, is dropped upon a hard surface from a height of 2 ft., it will not crack or flatten to more than half its original thickness. This determination is extremely simple, easy to make, may be readily distinguished, is suitable for moulding, and, in the hands of an experienced operator, is extremely accurate.

Method of Mixing.—The proportions of cement, sand and water should be carefully determined by weight, the sand and cement mixed dry, and the water added all at once. The mixing must be rapid and thorough, and the mortar, which should be stiff and plastic, should be firmly pressed into moulds with a trowel, without ramming, and struck off level. It will be found that if the mixing be rapid, it need only continue for about one

Fig. 13.—Improved Form of Fairbanks Testing Machine.

and one half minutes. The mixing should be done upon a glass or slate slab, and the hands should be protected by rubber gloves.

Storing Briquettes.—It is customary to store the briquettes immediately after making in a damp atmosphere for 24 hours. They are then immersed in water until tested. The reason for this is to secure uniformity of setting, and to prevent the drying out too quickly of the cement, thereby preventing shrinkage, cracks and greatly reducing the strength. To keep the samples damp when a suitable closet is not available, the briquettes are sometimes covered with a wet cloth having its ends dipped in water.

Testing Machines.—A large variety of testing machines is on the market, all of which are quite expensive. Any one of them will, if properly used, give satisfactory results. Figure 13 shows a cut of the improved Fairbanks machine, which will prove very satisfactory for ordinary testing purposes.

The method of operation of this machine is as follows: The briquette is placed in the clamps, and adjustment is made by the hand wheel P until the indicators are in line. By means of the hook lever Y, the worm is engaged with the gear; the shot valve is opened, allowing the shot to run into the bucket, the crank is turned with sufficient speed to hold the beam in equilibrium until the briquette is broken.

After the specimen has broken, the cup with its shot is removed and hung on the hook under the large ball, and the weight of the shot as given on the graduated beam shows the number of pounds required to break the specimen. A home-

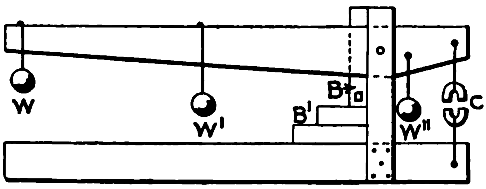

Fig. 14.—Home-Made Cement Testing Machine.

made testing machine of low cost, devised by F. W. Bruce, is shown in Fig. 14.

This machine consists of a counterpoised wooden lever, 10 ft. long, working on a horizontal pin between two broad uprights 20 ins. from one end. Along the top of the arm runs a grooved wheel carrying a weight. The distance from the fulcrum in feet and inches is marked on the surface of the lever. A clip for tensile tests is suspended from the short arm, 18 ins. from the fulcrum. The lower clip is fastened to the bed plate. The rail on which the wheel runs is a piece of light T-iron, fastened on top of the lever. The pin is iron, and the pin holes are reinforced with iron washers. The clamps are wood, and are fastened by clevis joints to the lever arm and bed-plate respectively. With care, very uniform results may be obtained with this machine.

Strength of Cement Mortars.—In making tensile tests the primary object is to ascertain the strength which will develop within a certain time. By determining the gain in strength between different dates of testing, some idea may be obtained of the ultimate strength which the cement will attain. In no case should a cement decrease in strength. Again, it should be remembered that the strength shown by a single test may not necessarily give a true indication of the value. Too high values for the one-day and seven-day neat cement tests should be regarded with suspicion, as the great strength is probably due to high liming, and the cement will after a time lose much of its early strength.

Specifications for tensile strength of cement usually stipulate that the materials must pass a minimum strength requirement at 7 and 28 days. The limit set is often so low that all but the poorest cements easily fulfill the requirement. W. Purves Taylor states that "the proper grounds for the judgment of the tests of tensile strength are four in number: (1) That both neat and sand briquettes shall pass a minimum specification at 7 and 28 days; (2) that the neat value at 7 days shall not be excessively great; (3) that there shall be no retrogression in the neat strength between 7 and 28 days, and (4) that the strength of the sand briquettes between these periods shall increase at least 10 or 15 per cent." It must be remembered that the sand test is the true criterion of strength, and no cement failing to pass this test should be accepted, even though the neat tests are satisfactory. If, however, the sand tests pass, and the neat fail, it may at times be justifiable to use the material if it passes the tests for soundness satisfactorily. Mr. Taylor also gives the following rules for accepting or rejecting material on the results of tensile tests:

"At 7 days:—Reject on decidedly low sand strength. Hold for 28 days on low or excessively high neat strength, or a sand strength barely failing to pass requirements."

"At 28 days:—Reject on failure in either neat or sand strengths. Reject on retrogression in sand strength, even if passing the 28 day requirements."

"Reject on retrogression in neat strength, if there is any other indication of poor quality, or if the 7-day test is low; otherwise accept."

"Accept if failing slightly in either neat or sand at 7 days and passing at 28 days."

A first-class cement when tested should give approximately the following values for tensile strength per square inch:

PORTLAND CEMENT.

Neat.

Age.		Strength.
24 hours	(in moist air)	175 lbs.
7 days	(1 day in moist air, 6 days in water)	500 "
28 days	(1 day in moist air, 27 days in water)	600 "

One Part Cement, Three Parts Sand.

Age.		Strength.
7 days	(1 day in moist air, 6 days in water)	170 lbs.
28 days	(1 day in moist air, 27 days in water)	240 "

NATURAL CEMENT.

Neat.

Age.		Strength.
24 hours	(in moist air)	40 lbs.
7 days	(1 day in moist air, 6 days in water)	125 "
28 days	(1 day in moist air, 27 days in water)	225 "

One Part Cement, Two Parts Sand.

Age.		Strength.
7 days	(1 day in moist air, 6 days in water)	75 lbs.
28 days	(1 day in moist air, 27 days in water)	140 "

SLAG CEMENT.

Neat.

Age.		Strength.
7 days	(1 day in moist air, 6 days in water)	350 lbs.
28 days	(1 day in moist air, 27 days in water)	500 "

One Part Cement, Three Parts Sand.

Age.		Strength.
7 days	(1 day in moist air, 6 days in water)	140 lbs.
28 days	(1 day in moist air, 27 days in water)	220 "

Myron S. Falk states* that cement and cement mixtures attain a strength not differing greatly from the ultimate strength within a period of three months from the time of setting, and practically within a month or so after this period no appreciable change of strength takes place.

Compressive Strength of Neat Cement.—The compressive strength of Portland cement bears a varying ratio to its tensile strength. The compressive strength usually increases faster than the tensile strength, but this ratio does not vary much from a fixed quantity, which may be taken as 10. Table I., taken from the Watertown Arsenal Report of 1902, gives values of the ratio between tensile and compressive strengths of neat cement mortars. Ten specimens of each kind were tested with varying percentages of water and at different ages.

*Cements, Mortars and Concrete, N. Y., 1905.

TABLE I.

Age in Air. Days.	Age in Water. Days.	Gauged with 20 per cent. water. Compressive strength in lbs. per sq. in.	Tensile strength in lbs. per sq. in.	Ratio.	Gauged with 22 per cent. water. Compressive strength in lbs. per sq. in.	Tensile strength in lbs. per sq. in.	Ratio.	Gauged with 25 per cent. water. Compressive strength in lbs. per sq. in.	Tensile strength in lbs. per sq. in.	Ratio.
1	..	717	196	3.7	595	189	3.1	430	190	2.3
7	..	3,040	354	8.6	3,260	392	8.3	2,610	402	6.5
28	..	3,990	566	7.1	3,760	457	8.2	3,130	450	7.0
1	6	4,250	780	5.5	4,720	666	5.8	3,880	329	11.8
1	27	7,370	906	8.1	6,870	866	7.9	7,580	758	10.0

The shearing strength of cement is somewhat greater than the tensile strength. According to Bauschinger, in the Proceedings of the Munich Technical Institute, its value is from 1.03 to 1.57 that of the tensile strength. A value of 1.25 times the tensile strength may be taken as a safe average value for the shearing strength of cement mortar.

Chemical Composition of Cement.—The chemical composition is one of the most important guides in determining the character of a cement, and an analysis should always be carefully made when large quantities are to be used. The proportion of magnesia and anhydrous sulphuric acid, and of soluble silica and alumina, to the lime should be determined. It is customary for cement chemists to grind up as fine as possible a given sample of cement, then determine the percentages of silica, alumina, lime, etc. A proper method of analysis would seem to be to determine the character of the cement as it is used without pulverizing or otherwise changing its physical character. The free silica should be separated from the mixture and the proportion of combined silica carefully determined, for it alone is an active agent, the free silica being inert and acting only as so much free sand.

The following table shows both the chemical composition as it is usually given, and as shown by an analysis, in which the free and combined silicas are separated:

1	2	3		4	5
Silica	SiO_2	21.90		Soluble (SiO_2)	18.45
Alumina	Al_2O_3	7.89	10.98	Alumina and Iron Oxides $Al_2O_3 + Fe_2O_3$	9.46
Iron Oxides	Fe_2O_3	3.09			
Lime	CaO.	62.04		CaO	61.89
Magnesia	MgO.	2.33		MgO	1.78
Sulphuric Oxide	SO_3	1.49		SO_3	1.87
				SiO_2 Insoluble in 10% HCl.	4.38

Column 3 shows the average composition of 11 well-known American cements, and column 5 shows the average composition of 7 high-class American cements.

Gypsum is sometimes added to a cement to increase its time of setting. When gypsum is used, an excess of lime is sometimes added to hide it. The addition of a solution of carbonate of soda to a cement adulterated with gypsum will again cause it to set quickly.

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CHAPTER III.

SAND, BROKEN STONE AND GRAVEL.

The sand and stone in concrete are called the aggregate.

It is necessary that the aggregate for reinforced concrete be selected with great care, for no matter how much or how good the cement, if the aggregate be of weak or inferior material, the concrete will be of poor quality. The materials which may be easily obtained near the locality in which the structure is to be erected are often not of the best; but, if properly used, will prove satisfactory.

Tests should be made to determine the strength of concrete made of the available materials; and if the strength thus determined is used in the design, a satisfactory structure may often be secured, the cost of which will be considerably less than when first-class materials brought from great distances are used. Sometimes cinder is used in place of the broken stone or gravel for light weight floors.

Definition of Mixtures.—When mixtures of cement, sand, gravel or broken stone are proportioned by volume, it is customary to designate them in multiples of the cement, which is taken as the unit of measure. Thus a 1 : 2 : 5 mixture consists of 1 part by volume of cement, 2 parts of sand and 5 of broken stone or gravel. A 1 : 3 mixture consists of 1 part of cement by volume to 3 parts of sand. When the sand and gravel are mixed together and not screened, as is often the practice in Europe, the mixture is spoken of as a 1 : 3 or 1 : 4 mixture, whichever it may be as in speaking of mortars.

Sand: Size and Shape of Grains.—Sand is used to fill the voids in the stone or gravel aggregate, and reduce the amount of cement required. The usual specifications for sand require that it shall be clean, sharp, coarse, and free from loam, clay and all vegetable matters. However, it is not essential that the sand be sharp and angular. The highest tests with cement have been obtained with sand having rounded grains with a dull surface. The rounded grains pack more closely than the angular grains, thus giving a smaller percentage of voids.

To secure a minimum of voids, a mixed size of grain from fine to coarse should be used. Such a sand is better than one having grains of uniform size, and gives as great or greater strength than a coarse sand. A fine sand does not give as great strength as coarse or mixed sized grains. Sand of uniform size and fine enough to pass a No. 40 sieve gives about 20 per cent. less strength than the larger sizes. There is no appreciable variation in strength when using different sized sands whose grains will not pass a No. 40 sieve. Hence it is not always essential that the sand be extremely coarse.

Effect of Loam in Sand.—It has been recognized by engineers for a number of years that the presence of moderate quantities of clay or loam in sand or gravel has no injurious effect on mortars and concrete. Recent tests seem to confirm this opinion. A series of tests made by J. C. Hain, Assoc. M., Am. Soc. C. E., show that sand containing loam is equal, or superior to, clean sand. Tests were made on 1:2 and 1:3 mortar, comparing clean sand with sands containing 2, 5, 10, and 20 per cent. of loam. A 1:2 mortar of clean sand gave slightly better and more uniform results. The 1:3 mortar, with sand containing up to 20 per cent of loam by weight, gave as high averages as clean sand, but the results were not as uniform as the latter. Tests were also made with sand from different pits, and containing from 2.5 to 7.7 per cent. of loam and clay. The sands containing the highest percentages of impurities gave the best results.

Prof. Sherman, Ohio State University (Eng. News, Nov. 19, 1903), reports tests on 1:3 cement mortars made with various percentages of clay and loam up to 15 per cent. of the sand, and states that of 72 tests, only 5 fell below the tensile strength of mortar containing no impurities. He concludes that clay or loam up to 15 per cent. is beneficial to cement mortars. Hence we may conclude that clay or loam in moderate quantities will not be injurious to mortar or concrete, if the concrete be thoroughly mixed and wet. It will be well, therefore, to make tests of sands containing impurities and compare results with tests on washed and standard sands before deciding against the use of the former, when they may, on account of their proximity to the work, prove economic.

Care should be taken in the selection of sands to exclude all those which have come in contact with acid or alkali solutions,

TABLE II.

SHOWING EFFECT OF LOAM AND CLAY ON TENSILE STRENGTH OF CEMENT MORTAR.

Old Shipment.

	7 Days No. 1. Lbs.	7 Days No. 2. Lbs.	7 Days No. 3. Lbs.	28 Days No. 1. Lbs.	28 Days No. 2. Lbs.	28 Days No. 3. Lbs.
Sand and cement, 3 to 1....	170	166	190	240	240	260
	180	168	191	245	236	265
Sand with 5% loam..........	187	...	...	250	...	...
	183	...	...	245	...	...
Sand with 10% loam..........	...	165	...	...	241	...
	...	175	...	...	250	...
Sand with 15% loam..........	...	...	203	...	...	275
	...	...	210	...	...	275

New Shipment.

Test completed....................	Sept. 7th, 1904.		Nov. 5th, 1904.	Feb. 5th, 1905.
Age of briquette when broken......	7 days.	28 days.	3 mos.	6 mos.
	Strength Lbs.	Lbs.	Lbs.	Lbs.
Sand and cement, 3 to 1...........	210	316	350	339
	207	337	328	346
5% Clay	220	356	341	340
	218	343	336	334
5% Loam	208	367	309	316
	201	359	321	320
10% Clay	210	321	329	327
	213	330	336	334
10% Loam	200	369	320	330
	207	365	329	331
15% Clay	200	296	270	225
	208	301	260	220
15% Loam	202	365	321	319
	205	368	328	315
20% Clay	184	262	250	200
	189	267	258	216
20% Loam	220	370	250	230
	216	372	241	225
25% Clay	172	240	230	175
	166	251	216	160
25% Loam	221	373	239	222
	218	375	248	217
30% Loam	220	361	256	210
	225	350	250	216
35% Loam	205	300	239	220
	198	306	248	212
40% Loam	198	290	240	198
	189	279	232	213

Effect of Clay and Loam.—The foregoing tests (Table II) are from the report on Defences of Galveston, Texas, by Capt. Edgar Jadwin, Corps of Engineers. (See Report of Chief Engineers, U. S. A., for 1905.)

The cement used was "Double Anchor" German brand; the sand, standard quality; the clay was taken from the cutter of a dredge working in Galveston Channel; the loam was heavy black soil from the main land. Both loam and clay were thoroughly pulverized, free apparently from all vegetable matter and sand, and sifted to remove lumps. All briquettes were made from one sample on the same day under the same conditions. The clay acted so unsatisfactorily during the working of the 25 per cent. batch that no more briquettes were made for this particular test, but the loam was continued to 40 per cent. Two shipments of cement were employed in the tests.

As will be seen, the loam mixtures retained their strength up to 35 per cent. for 7 and 28 days and 3 months, but for 6 months tests, appear to lose their strength when more than from 10 to 15 per cent. of loam was used. The clay mixtures decreased in strength when more than 10 to 15 per cent. of clay was used. For lower percentages than these in almost all cases the mixtures containing impurities were stronger than the clean 1 : 3 sand mixture.

Effect of Coal in Sand.—In the construction of the Harrisburg sewer (see index under Sewers), the sand used for the concrete was dredged from the Susquehanna River near by, and contained from 12 to 18 per cent. of fine anthracite coal. A series of special tests was made on this sand to determine the effect of the presence of the coal on its tensile strength. The sand was first washed and screened to remove the coal, only that passing a No. 24 sieve, and retained on a No. 30, being used. The coal thus removed was likewise screened, and that passing a No. 10 sieve, and retained on a No. 24, was used. All the cement used for these tests was taken from the same bag of Lehigh Portland cement, which gave the following strength neat and mixed 1 to 3 with standard sand:

1 day, neat	354 lbs.
7 days, neat	686 "
7 days, standard sand 1 : 3	183.5 "
28 days, standard sand 1 : 3	272 "

The test briquettes were made up of one part Lehigh Portland cement to three parts of sand, the sand containing varying percentages of coal from 0 to 100 per cent. It was found that there was no apparent decrease in strength when from 0 to 28 per cent. of coal was mixed with the sand, but there was a gradual diminution in strength as more coal, up to 100 per cent., was added. The final strength for 100 per cent. of coal was about one-fifth of the strength of the clean sand mixture.

Sand Washing.—When only dirty sand is available, and clean sand can only be obtained at a high cost, the dirty sand may be washed. When the quantity of sand to be used is not large, the washing may be done with a hose. A tank may be built about 8 ft. wide and 15 ft. long, with a bottom having a total slope of about 8 ins. in its length. The sides should be about 8 ins. high at the lower end, and increase gradually to a height of 3 ft. at the upper end. The lower end of the tank should be closed with a gate about 6 ins. high, sliding in guides, so that it can be removed. About 3 cu. yds. of the dirty sand are dumped on the upper end of the platform, and a stream of water from a ¾-in. hose played upon it, the man standing at the outside of the tank near its lower end. The water and sand flow down the platform, and the dirt passes off with the overflow of water over the gate. It will take about an hour to wash the 3 cu. yd. batch of sand. If two platforms are used the washing may be continuous. Halbert P. Gillette states,* that, when the operation is continuous, one man can wash 30 cubic yards a day at a cost of 5 cents per cubic yard for his labor. The cost of shoveling and extra hauling, due to the location of the washer, must be taken into account. When the water is pumped, about 10 cents more per cubic yard will be spent for coal and wages, making a total of about 25 cents per cubic yard.

When large quantities of sand are to be washed, expensive machinery of special design is used, and greatly reduces the cost of washing. Mr. H. W. Roper states that the cost of washing sand for U. S. Lock No. 3, at Springdale, Pa., with a specially designed washer, was 7 cents per cubic yard.

Concrete mixers are often used for washing sand, it being dumped into the machine in the usual manner. Water is then turned on, and when it overflows at the discharge end

*"Hand Book of Cost Data." New York, 1905.

the machine is started. The dirt is separated from the sand by this operation, and is carried off by the overflow of water. When the water runs clear, the washing is completed, and the sand is dumped in the usual manner.

Cost of Sand.—The cost of sand varies with the locality. The prevailing price at which sand is sold in New York City averages $1.00 per cubic yard delivered at the work. The items which go to make up the cost of sand are: (1) Cost of loading in the pit; (2) cost of hauling in wagons; (3) cost of freight; (4) cost of rehandling; (5) cost of screening and washing when necessary; and (6) cost of pit charges, or pit rental. The following data are furnished by Gillette.* Cost of loading into wagons will average about 10 cents per cubic yard for either sand or broken stone, wages being 15 cents per hour.

The cost of hauling in wagons may be taken at 28 cents per cubic yard, per mile, wages of team and driver being 35 cents per hour. Freight rate must be obtained for each individual case. The cost of rehandling will be as much (or more, depending upon conditions) as the original cost of loading. Screening necessitates an additional handling at a slightly greater cost, as the sand is thrown against an inclined screen. The cost of washing, as stated above, may be taken at the outside at 25 cents per cubic yard. The above data, together with the cost of sand in the pit, will enable an estimate to be made of the cost of sand in each individual case.

Stone Dust vs. Sand.—It was formerly supposed that the presence of stone dust in mortars and concretes was not only undesirable, but injurious. The dust was therefore screened out and replaced by sand. Numerous tests, made during the past few years, show that mortars containing stone dust are almost always superior in strength to those made of sand. Harry Taylor, M. Am. Soc. C. E., Capt. Corps of Engineers, U. S. A., tested 1,650 briquettes of 1 : 3, 1 : 4, and 1 : 5 mortars at 1, 3, 6 and 12 months, using crusher dust, standard crushed quartz and Plum Island sand. The briquettes made with crusher dust had an average strength 72 per cent. greater than crushed quartz briquettes, and 2.3 times greater than Plum Island sand briquettes. A 1 : 5 mixture with stone dust proved stronger than a 1 : 3 mix-

*"Hand Book of Cost Data."

ture with crushed quartz. Many other tests might be cited which show results equal to or greater than those quoted above.

Capt. John S. Sewell, Corps of Engineers, U. S. A., states that while using crushed gneiss the dust was found to contain minute flakes of mica, which when wet behaved like quick-sand, and when used in any quantity "killed" the cement so that it hardly set at all. This is a rare case, however, and undoubtedly in the use of almost all kinds of stone the dust can be employed with economy and no loss of strength.

Stone and Gravel.—Either broken stone or gravel may be used in making concrete. Whichever material is used, it should be hard and free from soft particles and all impurities. The strongest concretes are made from the hardest stone, crushed flint, quartz and trap rock giving better results than sandstone or limestone. Limestone should not be used for concrete employed in the construction of fireproof buildings or structures liable to be subjected to fire, as there is danger of this material calcining when subjected to high temperatures. The writer has seen limestone concrete used for a fireproof floor which after being subjected to extreme heat was so thoroughly calcined that the mass remaining after the fire had the appearance and consistency of freshly burned lime.

Mixed sizes of stone should be employed, as, by their use, a minimum of voids is obtained, and less mortar is needed to fill them. If the material be uniformly graded, screening is not necessary. In fact, many competent engineers use unscreened stone entirely, not even excluding the dust from the crusher.

Thorough mixing, however, must be insisted upon, as it distributes the fine particles of dust throughout the mass, fills the voids of the aggregate and increases the strength.

Gravel vs. Broken Stone.—Many engineers consider broken stone superior to gravel for concrete. Spencer B. Newberry states that "good quartz gravel is harder than any broken stone, except trap or quartzite, and owing to its rounded form contains much less voids than stone." There is no ground for believing that rounded stone or rounded sand gives less strength with cement than material composed of angular fragments. Certain natural sands, with nearly spherical grains, show much higher tests with cement than angular crushed quartz. A sufficient number of comparative tests of crushed stone and gravel concrete are not available, but the many examples of faultless work with cement, sand and gravel show that there is no need of going to a distance for costly crushed stone when gravel is available.

Mr. E. P. Goodrich, M. Am. Soc. C. E., states that he made comparative tests on a large number of 12-in. briquettes made of gravel and of broken stone concrete. Tests were also made on beams. The briquettes and beams of gravel and broken trap were prepared in a similar manner and broken as nearly as possible under identical conditions. It was found that the average values of the strength for the gravel concrete was higher than for the concrete made from broken trap rock. In many cases quartz pebbles in the gravel concrete were broken, while the angular stones of the broken stone concrete were not.

The specifications of the New York Rapid Transit Railway Commission, Contract No. 2, for concrete and reinforced concrete, permit the use of either screened gravel or broken stone. The proportions used for roof and sidewalls are 1 cement, 3 sand and 4 broken stone or gravel. Gravel is extensively used by European engineers.

Gravel should be screened when the concrete is to be used in a structure where accuracy of proportion is important.

Broken stone should be of hard, close grained quality, clean and free from argillaceous matter.

In reinforced concrete the broken stone or screened gravel for the concrete surrounding the reinforcement ought never to be larger than will pass a ½-inch screen when the reinforcement is small, or spaced closed together, or when placed near the surface. When larger sections are employed the stone may be increased in size, but should not exceed what will pass a 1¼-inch screen. The specifications of the New York Rapid Transit Railway Commission limit the size of the broken stone for reinforced concrete work to that which will pass a 1-inch screen. It is common practice to specify that all the stone or gravel shall pass a ¾-inch screen.

Ashes, Cinder and Coke Aggregates.—These aggregates are lighter than broken stone. Nails may be driven into them, and they may be easily cut or chipped. Their great porosity causes concretes made with these aggregates to be poor conductors of sound and heat. They are, therefore, good materials for fireproofing purposes. Care should be taken to select ashes which have been thoroughly burned. Plenty of water should be used in mixing these concretes, and no ramming in depositing them should be allowed, as they will be crushed thereby. This class of concrete is mainly employed in the expanded metal and Matrai systems, and for a large variety of other floor systems, which are used for filling between steel beams and girders employed in the construction of fireproof floors. It should be remembered, however, that concretes constructed of these materials possess much

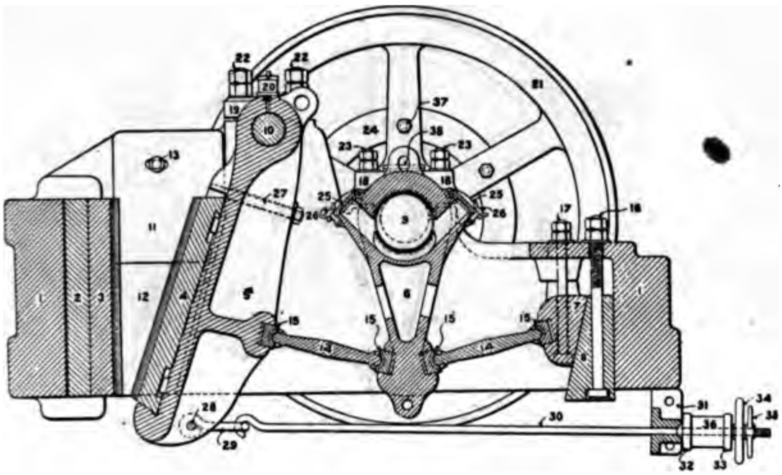


Fig. 15.—Jaw Crusher of the Farrel Type.

less strength than when stone or gravel is used, and their use avoided where great strength is required.

Crushing Stone.—Stone may be broken either by hand or by machinery. The economy of machine crushing makes the latter process almost universal. Two types of crushers are in common use: (1) The jaw or reciprocating crusher, sometimes called the Blake crusher, from the name of the original inventor; and (2) the gyratory crusher, called the Gates crusher, from the name of the inventor of this type.

The jaw crusher consists of a strong iron frame, near one end of which is a movable jaw. This jaw is moved backward and forward a short distance by means of a toggle joint and eccentric. As the jaw recedes the opening increases and the stone descends; as it returns toward the frame the stone is crushed and drops down as the jaw again recedes. The size of the largest pieces of crushed stone is determined by the distance between the jaw plates at their lower edge. Figure 15 shows a common form of the Blake crusher. In this machine the size of the stone is regulated by raising or lowering the wedge shown at the right hand side of the figure. It may also be regulated by changing the size of the toggles.

In the rotary or Gates crusher the reciprocating motion of the

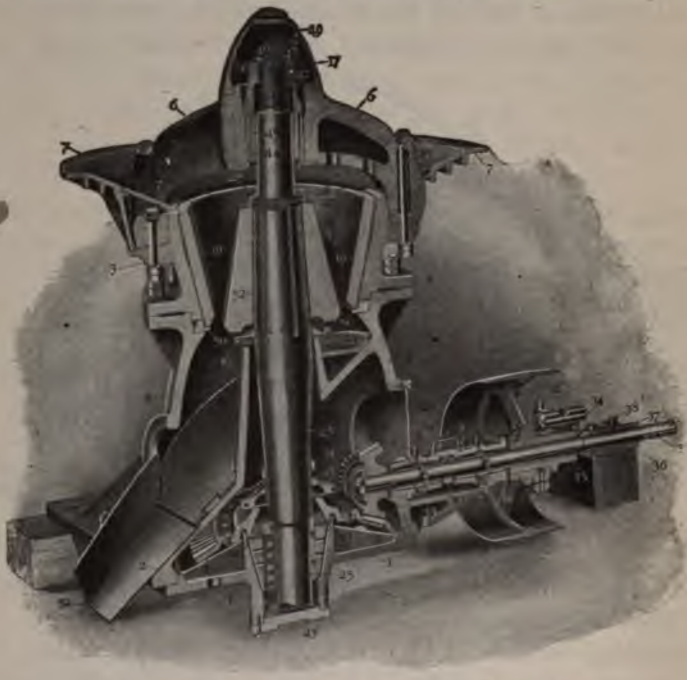


Fig. 16.—Gyratory Crusher of the Gates Type.

movable jaw is replaced by a gyratory motion of the movable jaw within a conical or hopper-shaped fixed jaw. In this type of crusher the movable jaw usually consists of a cone suspended at the top, and caused to gyrate within the lower enveloping section constituting the lower jaw. The space between the two jaws converges downward and is annular in shape. The rocking and rotary motion of the movable jaw within the walls of the cavity causes it to be constantly approaching the fixed jaw on one side, while receding from it on the other. The stone is crushed as it descends, and the breaking of the rock takes place continuously in some part of the annular space between the jaws. The size of the product is regulated by raising or lowering the movable jaw. The continuous crushing is one of the advantages of the Gates type of crusher over the Blake crusher, for in the latter the stone is only crushed while the jaws approach each other. A disadvantage of this type consists of its great weight and consequent lack of portability. Figure 16 shows a section of the Gates or gyratory crusher.

The highest economy in stone crushing will be found to result from a proper location and arrangement of the plant. The essential points to be considered are: (1) the location of the feed-

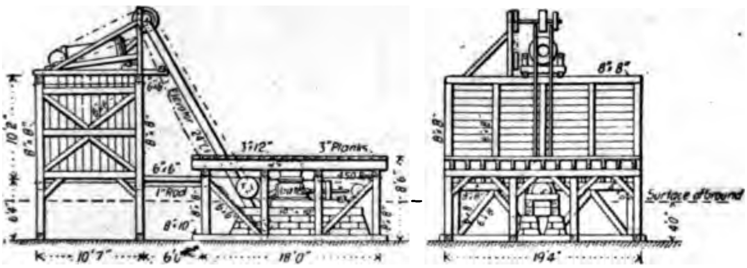


Fig. 17.—Crushing Plant with Elevator.

ing platform at the level of the top of the hopper or jaws; and (2) the dumping platform, or, when a screen is used, the location of the screen below, the discharging spout of the machine with ample facilities for the quick removal of the crushed product. This arrangement may be had on a hillside. When the plant must be located on level ground an elevator is used to raise the broken stone from the dumping platform to the mouth of the screen (see Fig. 17). Figure 18 shows crushing plant in which the elevator dumps the stone directly into the bin.

Cost of Stone Crushing.*—The cost of breaking stone by hand will average from 50 cents to $1.00 per cubic yard, depending upon the hardness of the stone. A skilled man should break 3 cubic yards of limestone in a 10-hour day. The cost of crushing by machinery depends upon conditions, and will average from 25 to 50 cents per cubic yard. Where stone must be quarried, it will be found that the quarry expense will also average 25 to 50 cents per cubic yard.

*For detailed information on the cost of quarrying and crushing, see Gillette's "Hand Book of Cost Data."

The price of broken stone depends upon the locality. It seldom is less than 75 cents per cubic yard delivered at the work, and is usually about $1.25, but it may be as much as $2.50, or even

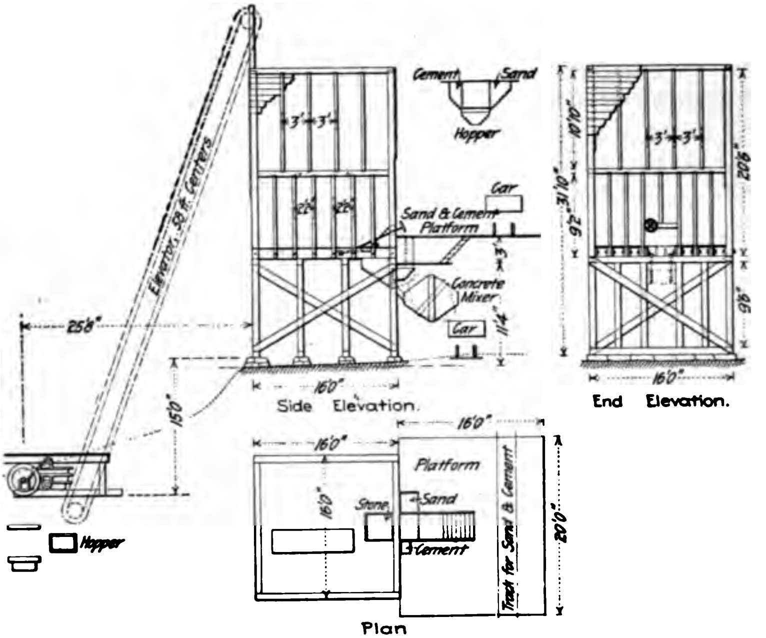


Fig. 18.—Stone Crushing and Concrete Mixing Plant.

more in rare cases. Gravel is generally cheaper than broken stone.

Screening Stone or Gravel.—When it is necessary to screen sand, stone or gravel, it may be done either by hand, the opera-

Fig. 19.—Rotary Screen.

tion simply consisting of shoveling the material against an inclined screen, or by means of large screens inclined at an angle varying from 35 to 45 degrees, the materials being dumped by machinery upon the screen; or rotary screens may be used. The latter method will generally be found to be the cheapest.

The rotary screen is shown in Fig. 19. Holes of different sizes are punched in the circumference of the screen, the smallest holes being near the upper end. This enables the smaller sizes to be screened out first, and the larger stone remains until it reaches the lower end of the screen.

The cost of screening will average from 3 to 8 cents per cubic yard when the rotary screen is used, and from 8 to 15 cents when the screening is done by hand. It will be found that the largest cost items will be those chargeable to handling the materials. Hence the plant should be designed to obtain the highest economy of labor.

CHAPTER IV.

PROPORTIONING CONCRETE.

In proportioning to obtain an ideal concrete, the materials composing the aggregate should be of graded sizes from the largest pieces of stone down to the finest particles of sand. Such proportions of each size should be used as to produce the least voids, hence the most compact mass. The mass may then be solidified by mixing with it such an amount of finely ground cement as will thoroughly coat each and every particle of sand and gravel or broken stone with a film of cement paste, and also entirely fill all small voids remaining after the materials have been thoroughly mixed.

It is evident that, on account of practical considerations, it will be impossible to secure an ideal concrete, but only to approximate it. As a uniformly graded aggregate can not be had, it is customary at times to limit the size of the stone, screening out everything above and below a certain size. The amount of voids in the stone is then determined, and enough sand used to fill them, and sufficient cement to coat over all particles of sand and stone and fill all remaining voids. This operation is called proportioning the ingredients, and a mixture so proportioned is said to be a well-balanced mixture. The operation is sometimes called balancing. When the proportions of the ingredients are such as to have all voids filled and each and every particle of the aggregate covered with a film of cement, a concrete of maximum strength will be secured, regardless of the relative proportions.

The amount of water necessary to secure the proper consistency must also be determined. The materials are then mixed and put in place.

Proportioning Concrete for Different Uses.—The proportions of the ingredients used will depend largely upon the nature of the work. When great strength or a high degree of impermeability is required, a concrete very rich in cement should be used. For many purposes, neglecting the question of cost, a concrete not so rich in cement will fill all requirements, and is more desirable as there is less liability of a change in volume, the expansion and contraction on hardening being due entirely to the cement. Some constructors use concrete with a varying amount of cement, and, therefore, of varying strength in different parts of the same piece of concrete. A lean concrete is used in the tensile part of the piece, and a concrete very rich in cement is used where the compression strains are the highest. This is a dangerous procedure, as there is a tendency to form planes of cleavage at the junction of two concretes of varying richness. Much better results will be secured if a homogeneous concrete is used throughout the whole piece.

For beams and slabs having small thickness, a cement and sand mixture alone is employed; and, for pieces of larger dimensions, it has been European practice to use a cement and gravel mixture, the gravel being of moderate size and containing from 25 to 50 per cent. of sand. Two methods of proportioning are employed by European engineers, viz., by weight and by volume. The latter method only will be considered in this book, as it is almost universally used by American engineers. The usual proportions employed for slabs, arches, floors, etc., are from 1 : 1½ to 1 : 3½ or 1 : 4½ cement and gravel; with sand mixtures the most common proportions are 1 : 3 or 1 to 3½.

For pipes, Bonna uses almost exclusively 1 cement to 1.8 gravel and sand. Considère recommends for hooped columns, 1 cement, 0.7 sand and 2.05 broken stone. Concrete mixtures to replace the sand and gravel mixtures are coming into greater favor among European engineers.

In America broken stone concrete having proportions of 1:2:4, 1:3:5, and 1:3:6 are most commonly used, although at times 1:2:3, 1:1½:4 and 1:2½:5 mixtures are preferred by some engineers.

The character of the materials and the use for which they are to be employed are the determining factors in choosing any given mixture. For impervious concrete a mixture rich in cement is used, a 1:1½ or 1:2 mortar giving excellent results. Mr. Newman states that a 1:1½ mortar will resist a 75-foot head of water. The most satisfactory concretes will result when a well balanced concrete is secured, regardless of what the relative proportions may be. Thus at 1 : 3 : 5 well-balanced mixture may be stronger than a 1 : 2 : 3 or 1 : 2 : 4 poorly balanced mixture.

Filling the Voids.—By using a mixture composed of materials of varying sizes the voids are much less than in a material composed of pieces of uniform size. Sand is used to fill the voids of the stone, and cement to fill those of the sand, and if the proportions used are correct the resulting mass is practically a solid. In order to know the amount of sand and cement necessary to fill the voids, a metal box of known cubical capacity, preferably 1 cubic foot, should be supplied. This is weighed, filled with the material, thoroughly shaken down and again weighed. The difference between the weight of the material filling the box and the weight of a cubic foot of solid stone, represents the amount of the voids. Solid quartz or limestone without voids weighs 165.4 lbs. per cubic foot. If the broken stone weighs 99 lbs., the voids will be 66.4 lbs. or 40 per cent. Therefore, theoretically 0.4 of a cubic foot of sand should be used to fill the voids in the stone. The voids in the sand may be determined in the same manner.

Let us assume that the voids in the sand are 38 per cent., then to make 1 cubic foot of concrete it will take $0.38 \times 0.4 = 0.152$ cubic feet of cement, and we will theoretically require 0.152 cu. ft. cement + 0.4 cu. ft. sand + 1.0 cu. ft. broken stone to make 1 cubic foot of concrete, or a 1 : 2.63 : 6.58 mixture. In order to secure good results, a slight excess of cement over the theoretical requirements is needed. Mr. William B. Fuller's rule is: "Add cement as economy dictates up to 10 per cent. in excess of the voids in the combined materials." It will be found, however, that the proportions of the ingredients necessary to make a given quantity of concrete will vary somewhat from the amount as determined by the voids. This may be due to the condition of the stone or sand as regards moisture or compactness when measured, or whether the cement is packed or loose. The method of mixing used, and the thoroughness with which it is done, as well as the manner in which the concrete is placed and the amount of tamping done upon it, may modify somewhat the amount of space that given quantities of materials will occupy in the finished work.

Table III. is given by Mr. Edwin Thacher from experiments made by him for volumes on cement, sand, gravel, broken stone, mortar and concrete. The original volumes of all materials were measured loose, but gently shaken down.

TABLE III.

VOLUMES OF VARIOUS MATERIALS.

Cement.	Volume of loose cement.	Water added by measure.	Volume of stiff cement paste.	Remarks.
Portland cement (Atlas)..	1.00	0.35	0.78	6.56 barrels of cement =
Natural cement, Louisville	1.00	0.43	0.78	1 cu. yd. measured loose.

Aggregate.	Volume loose.	Solids.	Voids.
1. Sand, moist, fine, will pass 18-mesh sieve.	1.00	0.57	0.43
2. Sand, moist, coarse, will not pass 18-mesh sieve	1.00	0.65	0.35
3. Sand, moist, coarse and fine mixed (ordinary)....	1.00	0.62	0.38
4. Sand, dry, coarse and fine mixed.................	1.00	0.70	0.30
5. Stone screenings and stone dust..................	1.00	0.58	0.42
6. Gravel, ¾ in. and under, 6 per cent coarse sand...	1.00	0.67	0.33
7. Broken stone, 1 in. and under....................	1.00	0.54	0.46
8. Broken stone, 2½ in. and under, dust only screened out	1.00	0.59	0.41
9. Broken stone, 2½ in. and under, most small stones screened out	1.00	0.55	0.45

Mortars with No. 3 Sand.

Parts of sand mixed with 1 part of cement..........	1.0	1.5	2.0	2.5	3.0	3.5	4.0	5.0
Volume of slush mortar.....	1.40	1.78	2.17	2.55	2.98	3.39	3.82	4.65
Required for 1 cu. yd.—								
Cement, bbls.......	4.70	3.70	3.04	2.58	2.21	1.94	1.72	1.41
Sand, cu. yds.......	0.71	0.84	0.92	0.98	1.01	1.03	1.05	1.08
Volume of dry facing mortar (rammed)	1.22	1.57	1.93	2.28	2.64	2.99	3.35	4.08
Required for 1 cu. yd.—								
Cement, bbls.......	5.40	4.18	[illegible].41	2.88	2.49	2.20	1.96	1.61
Sand, cu. yds..	0.82	0.95	1.04	1.10	1.14	1.17	1.20	1.23

Tables IV and V, by Mr. Thacher, from tests made at Cornell University, are the results of a large number of experiments, and give the quantities of cement and aggregate necessary to make 1 cubic yard of concrete, the cement being measured loose.

No one table or set of tables will give correct results under the varying conditions met with in practice. Many formulas, some of them very intricate, have been proposed, but for practical use have little value. Probably as simple and rational formulas as any are those suggested by Mr. Halbert P. Gillette,* which we here give with tables showing their application:

Gillette's Formulas.—When loose sand is mixed with water, its volume or bulk is increased; subsequent jarring will increase its volume, but still leave a net gain of about 10%; that is, 1 cubic foot of dry sand becomes about 1.1 cubic feet of damp sand. Not only does this increase

*"Hand Book of Cost Data."

TABLE IV.
PROPORTIONS FOR PORTLAND CEMENT CONCRETE.
(Cement Measured Loose: 1 Barrel = 4.12 Cu. Ft.)

Mixtures			Required for 1 Cubic Yard Rammed Concrete					
			Stone 1 in. and under, dust screened out.			Gravel ¾ in. and under.		
Cement.	Sand.	Stone.	Cement, bbls.	Sand, cu. yds.	Stone, cu. yds.	Cement. bbls.	Sand, cu. yds.	Gravel, cu. yds
1	1.0	2.0	2.57	0.39	0.78	2.30	0.35	0.74
1	1.0	2.5	2.29	0.35	0.70	2.10	0.32	0.80
1	1.0	3.0	2.06	0.31	0.94	1.89	0.29	0.86
1	1.0	3.5	1.84	0.28	0.98	1.71	0.26	0.91
1	1.5	2.5	2.05	0.47	0.78	1.83	0.42	0.73
1	1.5	3.0	1.85	0.42	0.84	1.71	0.39	0.78
1	1.5	3.5	1.72	0.39	0.91	1.57	0.36	0.83
1	1.5	4.0	1.57	0.36	0.96	1.46	0.33	0.88
1	1.5	4.5	1.43	0.33	0.98	1.34	0.31	0.91
1	2.0	3.0	1.70	0.52	0.77	1.54	0.47	0.73
1	2.0	3.5	1.57	0.48	0.83	1.44	0.44	0.77
1	2.0	4.0	1.46	0.44	0.89	1.34	0.41	0.81
1	2.0	4.5	1.36	0.42	0.93	1.26	0.38	0.86
1	2.0	5.0	1.27	0.39	0.97	1.17	0.36	0.89
1	2.5	3.5	1.45	0.55	0.77	1.32	0.50	0.70
1	2.5	4.0	1.35	0.52	0.82	1.24	0.47	0.75
1	2.5	4.5	1.27	0.48	0.87	1.16	0.44	0.80
1	2.5	5.0	1.19	0.46	0.91	1.10	0.42	0.83
1	2.5	5.5	1.13	0.43	0.94	1.03	0.39	0.86
1	2.5	6.0	1.07	0.41	0.97	0.98	0.37	0.89
1	3.0	4.0	1.26	0.58	0.77	1.15	0.52	0.72
1	3.0	4.5	1.18	0.54	0.81	1.09	0.50	0.75
1	3.0	5.0	1.11	0.51	0.85	1.03	0.47	0.78
1	3.0	5.5	1.06	0.48	0.89	0.97	0.44	0.81
1	3.0	6.0	1.01	0.46	0.92	0.92	0.42	0.84
1	3.0	6.5	0.96	0.44	0.95	0.88	0.40	0.87
1	3.0	7.0	0.91	0.42	0.97	0.84	0.38	0.89
1	3.5	5.0	1.05	0.56	0.80	0.96	0.50	0.76
1	3.5	5.5	1.00	0.53	0.84	0.92	0.48	0.78
1	3.5	6.0	0.95	0.50	0.87	0.88	0.46	0.80
1	3.5	6.5	0.92	0.49	0.91	0.83	0.44	0.82
1	3.5	7.0	0.87	0.47	0.93	0.80	0.43	0.85
1	3.5	7.5	0.84	0.45	0.96	0.76	0.41	0.87
1	3.5	8.0	0.80	0.42	0.97	0.73	0.39	0.89
1	4.0	6.0	0.90	0.55	0.82	0.83	0.51	0.77
1	4.0	6.5	0.87	0.53	0.85	0.80	0.49	0.79
1	4.0	7.0	0.83	0.51	0.89	0.77	0.47	0.81
1	4.0	7.5	0.80	0.49	0.91	0.73	0.44	0.83
1	4.0	8.0	0.77	0.47	0.93	0.71	0.43	0.86
1	4.0	8.5	0.74	0.45	0.95	0.68	0.42	0.88
1	4.0	9.0	0.71	0.43	0.97	0.65	0.40	0.89
1	5.0	9.0	0.66	0.50	0.90	0.61	0.46	0.83
1	5.0	10.0	0.62	0.47	0.95	0.57	0.43	0.87

TABLE V.
PROPORTIONS FOR PORTLAND CEMENT CONCRETE.
(Cement Measured Loose, 1 Bbl. = 4.12 Cu. Ft.)

Mixtures			Required for 1 Cubic Yard Rammed Concrete — Stone 2½ in. and under, dust screened out.			Required for 1 Cubic Yard Rammed Concrete — Stone 2½ in. with most small stone screened out.		
Cement.	Sand.	Stone.	Cement, bbls.	Sand, cu. yds.	Stone, cu. yds.	Cement, bbls.	Sand, cu. yds.	Stone, cu. yds.
1	1.0	2.0	2.63	0.40	0.80	2.72	0.41	0.83
1	1.0	2.5	2.34	0.36	0.89	2.41	0.37	0.92
1	1.0	3.0	2.10	0.32	0.96	2.16	0.33	0.98
1	1.0	3.5	1.88	0.29	1.00	1.88	0.29	1.05
1	1.5	2.5	2.09	0.48	0.80	2.16	0.49	0.82
1	1.5	3.0	1.90	0.43	0.87	1.96	0.45	0.89
1	1.5	3.5	1.74	0.40	0.93	1.70	0.41	0.96
1	1.5	4.0	1.61	0.37	0.98	1.64	0.38	1.00
1	1.5	4.5	1.46	0.33	1.00	1.51	0.35	1.06
1	2.0	3.0	1.73	0.53	0.79	1.78	0.54	0.81
1	2.0	3.5	1.61	0.49	0.85	1.66	0.50	0.88
1	2.0	4.0	1.48	0.45	0.90	1.53	0.47	0.93
1	2.0	4.5	1.38	0.42	0.95	1.43	0.43	0.98
1	2.0	5.0	1.29	0.39	0.98	1.33	0.39	1.03
1	2.5	3.5	1.48	0.56	0.79	1.51	0.58	0.81
1	2.5	4.0	1.38	0.53	0.84	1.42	0.54	0.87
1	2.5	4.5	1.29	0.49	0.88	1.33	0.51	0.91
1	2.5	5.0	1.21	0.46	0.92	1.26	0.48	0.96
1	2.5	5.5	1.15	0.44	0.96	1.18	0.44	0.99
1	2.5	6.0	1.07	0.41	0.98	1.10	0.41	1.03
1	3.0	4.0	1.28	0.58	0.78	1.32	0.60	0.80
1	3.0	4.5	1.20	0.55	0.82	1.24	0.57	0.85
1	3.0	5.0	1.14	0.52	0.87	1.17	0.54	0.89
1	3.0	5.5	1.07	0.49	0.90	1.11	0.51	0.93
1	3.0	6.0	1.02	0.47	0.93	1.06	0.48	0.97
1	3.0	6.5	0.98	0.44	0.96	1.00	0.45	1.01
1	3.0	7.0	0.92	0.42	0.98	0.94	0.42	1.05
1	3.5	5.0	1.07	0.57	0.82	1.11	0.59	0.85
1	3.5	5.5	1.02	0.54	0.85	1.06	0.56	0.89
1	3.5	6.0	0.97	0.51	0.89	1.00	0.53	0.92
1	3.5	6.5	0.93	0.49	0.92	0.96	0.51	0.95
1	3.5	7.0	0.89	0.47	0.95	0.91	0.49	0.98
1	3.5	7.5	0.86	0.45	0.98	0.86	0.47	1.01
1	3.5	8.0	0.82	0.43	1.01	0.81	0.45	1.04
1	4.0	6.0	0.92	0.56	0.84	0.95	0.58	0.87
1	4.0	6.5	0.88	0.53	0.87	0.91	0.55	0.90
1	4.0	7.0	0.84	0.51	0.90	0.87	0.53	0.93
1	4.0	7.5	0.81	0.50	0.93	0.84	0.51	0.96
1	4.0	8.0	0.78	0.48	0.95	0.81	0.49	0.98
1	4.0	8.5	0.76	0.46	0.98	0.78	0.47	1.01
1	4.0	9.0	0.73	0.44	1.01	0.75	0.45	1.04
1	5.0	9.0	0.67	0.52	0.93	0.70	0.53	0.96
1	5.0	10.0	0.63	0.48	0.96	0.65	0.50	1.00

in the volume of the sand occur, but, instead of increasing the voids that can be filled with cement, there is an absolute loss in the volume of available voids. This is due to the space occupied by the water necessary to bring the sand to the consistency of mortar; furthermore, there is seldom a perfect mixture of the sand and cement in practice, thus reducing the available voids. It is safe to call this reduction in available voids about 10%.

When loose dry Portland cement is wetted, it shrinks about 15% in volume, behaving differently from the sand, but it never shrinks back to quite as small a volume as it occupies when packed tightly in a barrel. Since barrels of different brands vary widely in size, the careful engineer or contractor will test any brand he intends using in large quantities, in order to ascertain exactly how much cement paste can be made. He will find a range of from 3.2 cubic feet to 3.8 cubic feet per barrel of Portland cement. Obviously the larger barrel may be cheaper, though its price is higher. Specifications often state the number of cubic feet that will be allowed per barrel in mixing the concrete ingredients, so that any rule or formula to be of practical value must contain a factor to allow for the specified size of the barrel, and another factor to allow for the actual number of cubic feet of paste that a barrel will yield—the two being usually quite different.

The deduction of a rational, practical formula for computing the quantity of cement required for a given mixture will now be given, based upon the facts above outlined.

Let p = number of cu. ft. cement paste per bbl. as determined by actual test.
n = number of cu. ft. of cement per bbl. as specified in the specifications.
s = parts of sand (by volume) to one part of cement, as specified.
g = parts of gravel or broken stone (by volume) to one part of cement, as specified.
v = percentage of voids in the dry sand, as determined by test.
V = percentage of voids in the gravel or stone, as determined by test.

Then, in a mortar of 1 part cement to s parts sand we have:

$n\ s$ = cu. ft. of dry sand to 1 bbl. cement.
$n\ s\ v$ = cu. ft. of voids in the dry sand.
$0.9\ n\ s\ v$ = cu. ft. of available voids in the wet sand.
$1.1\ n\ s$ = cu. ft. of wet sand.
$p - 0.9\ n\ s\ v$ = cu. ft. of cement paste in excess of the voids.

Therefore:

$$1.1\ n\ s + (p - 0.9\ n\ s\ v) = \text{cu. ft. of mortar per bbl.}$$

Therefore:

$$N = \frac{27}{1.1\ n\ s + (p - 0.9\ n\ s\ v)} = \frac{27}{p + n\ s\ (1.1 - 0.9\ v)}$$

N being the number of barrels of cement per cu. yd. of mortar.

When the mortar is made so lean that there is not enough cement paste to fill the voids in the sand, the formula becomes

$$N = \frac{27}{1.1\ n\ s}$$

A similar line of reasoning will give us a rational formula for determining the quantity of cement in concrete; but there is one point of difference between sand and gravel (or broken stone), namely, that the gravel does not swell materially in volume when mixed with water. However, a certain amount of water is required to wet the surface of the pebbles, and this water reduces the available voids, that is, the voids that can be filled by the mortar. With this in mind the following deduction is clear, using the nomenclature and symbols above given:

$n\ g$ = cu. ft. of dry gravel (or stone).
$n\ g\ V$ = cu ft. of available voids in the wet gravel.
$0.9\ n\ g\ V$ = cu ft. of available voids in the wet gravel.
$p + n\ s\ (1.1 - 0.9\ v) - 0.9\ n\ g\ V$ = excess of mortar over the available voids in the wet gravel.
$n\ g + p + n\ s\ (1.1 - 0.9\ v) - 0.9\ n\ g\ V$ = cu. ft. of concrete from 1 bbl. cement.

$$N = \frac{27}{p + n\ s\ (1.1 - 0.9\ v) + n\ g\ (1 - 0.9\ V)},$$

N being the number of barrels of cement required to make 1 cu. yd. of concrete.

This formula is rational and perfectly general. Other experimenters may find it desirable to use constants slightly different from the 1.1 and the 0.9, for fine sands swell more than coarse sands, and hold more water.

The reader must bear in mind that when the voids in the sand exceed the cement paste, and when the available voids in the gravel (or stone) exceed the mortar, the formula becomes

$$N = \frac{27}{n\ g}$$

These formulas give the amounts of cement in mortars and concretes compacted in place. Tables VI to IX are based upon the foregoing theory, and will be found to check satisfactorily with actual tests.

TABLE VI.

BARRELS OF PORTLAND CEMENT PER CUBIC YARD OF MORTAR.

(Voids in sand being 35%, and 1 bbl. cement yielding 3.65 cu. ft. of cement paste.)

Proportion of Cement to Sand.	1 to 1.	1 to 1½.	1 to 2.	1 to 2½.	1 to 3.	1 to 4.
	Bbls.	Bbls.	Bbls.	Bbls.	Bbls.	Bbls.
Barrel specified to be 3.5 cu. ft....	4.22	3.49	2.97	2.57	2.28	1.76
Barrel specified to be 3.8 cu. ft....	4.09	3.33	2.81	2.45	2.16	1.62
Barrel specified to be 4.0 cu. ft....	4.00	3.24	2.73	2.36	2.08	1.54
Barrel specified to be 4.4 cu. ft....	3.81	3.07	2.57	2.27	2.00	1.40
Cu. yd. sand per cu. yd. mortar..	0.6	0.7	0.8	0.9	1.0	1.0

TABLE VII.

BARRELS OF PORTLAND CEMENT PER CUBIC YARD OF MORTAR.

(Voids in sand being 45%, and 1 bbl. cement yielding 3.4 cu. ft. of cement paste.)

Proportion of Cement to Sand.	1 to 1.	1 to 1½.	1 to 2.	1 to 2½.	1 to 3.	1 to 4.
	Bbls.	Bbls.	Bbls.	Bbls.	Bbls.	Bbls.
Barrel specified to be 3.5 cu. ft....	4.62	3.80	3.25	2.84	2.35	1.76
Barrel specified to be 3.8 cu. ft....	4.32	3.61	3.10	2.72	2.16	1.62
Barrel specified to be 4.0 cu. ft....	4.19	3.46	3.00	2.64	2.05	1.54
Barrel specified to be 4.4 cu. ft....	3.94	3.34	2.90	2.57	1.86	1.40
Cu. yd. sand per cu. yd. mortar..	0.6	0.8	0.9	1.0	1.0	1.0

In using these tables remember that the proportion of cement to sand is by volume, and not by weight. If the specifications state that a barrel of cement shall be considered to hold 4 cubic feet, for example, and that the mortar shall be 1 part cement to 2 parts sand, then 1 barrel of cement is mixed with 8 cubic feet of sand, regardless of what is the actual size of the barrel, and regardless of how much cement paste can be made with a barrel of cement. If the specifications fail to state what the size of a barrel will be, then the contractor is left to guess.

If the specifications call for proportions by weight, assume a Portland barrel to contain 380 pounds of cement, and test the actual weight of a cubic foot of the sand to be used. Sand varies extremely in weight, due both to the variation in the per cent. of voids, and to variation in the kind of minerals of which the sand is composed. A quartz sand having 35% voids weighs 107 pounds per cubic foot; but a quartz sand having 45% voids weighs only 91 pounds per cubic foot. If the weight of the sand must be guessed at, assume 100 pounds per cubic foot. If the specifications require a mixture of 1 cement to 2 of sand by weight, we will have 380 pounds (or 1 barrel) of cement mixed with 2 × 380 or 760 pounds of sand; and if the sand weighs 90 pounds per cubic foot, we shall have 760 ÷ 90, or 8.44 cubic feet of sand to every barrel of cement. In order to use the tables above given, we may specify our own size of barrel; let us say 4 cubic feet; then 8.44 ÷ 4 gives 2.11 parts of sand by volume to 1 part of cement. Without material error we may call this a 1 to 2 mortar, and use the tables, remembering that one barrel is now "specified to be" 4 cubic feet. If we have a brand of cement that yields 3.4 cubic feet of paste per barrel and sand having 45% voids, we find that approximately 3 barrels of cement per cubic yard of mortar will be required.

It should be evident from the foregoing discussions that no table can be made and no rule can be formulated that will yield accurate results unless the brand of cement is tested and the percentage of voids in the sand determined. This being so, the sensible plan is to use the tables merely as a rough guide, and, where the quality of cement to be used is very large, to make a few batches of mortar, using the available brands of cement and sand in the proportions specified. Ten dollars spent in this way may save a thousand, even on a comparatively small job, by showing what cement and sand to select.

TABLE VIII.

INGREDIENTS IN 1 CU. YD. OF CONCRETE.

(Sand voids 40%, stone voids 45%; Portland cement barrel yielding 3.65 cu. ft. paste. Barrel specified to be 3.8 cu. ft.)

	Proportions by Volume					
	1:2:4	1:2:5	1:2:6	1:2½:5	1:2½:6	1:3:4
Bbls. cement per cu. yd. concrete...	1.46	1.30	1.18	1.13	1.00	1.25
Cu. yd. sand per cu. yd. concrete...	0.41	0.36	0.33	0.40	0.35	0.53
Cu. yd. stone per cu. yd. concrete..	0.82	0.90	1.00	0.80	0.84	0.71
	Proportions by Volume					
	1:3:5	1:3:6	1:3:7	1:4:7	1:4:8	1:4:9
Bbls. cement per cu. yd. concrete...	1.13	1.05	0.96	0.82	0.77	0.73
Cu. yd. sand per cu. yd. concrete...	0.48	0.44	0.40	0.46	0.43	0.41
Cu. yd. stone per cu. yd. concrete..	0.80	0.88	0.93	0.80	0.86	0.92

This table is to be used where cement is measured packed in the barrel, for the ordinary barrel holds 3.8 cu. ft.

It will be seen that the above table can be condensed into the following rule: Add together the number of parts and divide this sum with ten; the quotient will be approximately the number of barrels of cement per cubic yard. Thus for 1 : 2 : 5 concrete, the sum of the parts is 1 + 2 + 5, which is 8, then 10 ÷ 8 is 1.25

barrels, which is approximately equal to the 1.30 barrels given in the table. Neither this rule nor this table is applicable if a different size of cement barrel is specified, or if the sand or stone differ materially from 40% and 45% respectively. There are such innumerable combinations of varying voids, and varying sizes of barrel, that the author does not deem it worth while to give other tables.

TABLE IX.

INGREDIENTS IN 1 CU. YD. OF CONCRETE.

(Sand voids 40%, stone voids 45%; Portland cement barrel yielding 3.65 cu. ft. of paste. Barrel specified to be 4.4 cu. ft.)

	Proportion by volume					
	1:2:4	1:2:5	1:2:6	1:2½:5	1:2½:6	1:3:4
Bbls. cement per cu. yd. concrete...	1.30	1.16	1.00	1.07	0.96	1.08
Cu. yd. sand per cu. yd. concrete...	0.42	0.38	0.33	0.44	0.40	0.53
Cu. yd. stone per cu. yd. concrete..	0.84	0.95	1.00	0.88	0.95	0.71
	Proportion by volume					
	1:3:5	1:3:6	1:3:7	1:4:7	1:4:8	1:4:9
Bbls. cement per cu. yd. concrete...	0.96	0.90	0.82	0.75	0.68	0.64
Cu. yd. sand per cu. yd. concrete...	0.47	0.44	0.40	0.49	0.44	0.42
Cu. yd. stone per cu. yd. concrete..	0.78	0.88	0.93	0.86	0.88	0.95

NOTE.—This table is to be used where the cement is measured loose after dumping into a box, for under such conditions a barrel of cement yields 4.4 cu. ft. of loose cement.

Voids in Sand.—The amount of voids in sand will depend upon the shape of the grains, the degree of uniformity in size of grains, the amount of moisture present and the amount of compacting to which the mass has been subjected. If all the grains in a given mass of sand are of uniform size the percentage of voids will be independent of the size of the grains. If, however, the grains be of varying sizes, the percentage of voids will be reduced. The mixture of a small amount of water with dry sand increases its bulk. with bank sand the greatest volume of voids per unit volume will be obtained when the percentage of water varies from 5 to 8 per cent.

For convenience, we will assume that sand is divided into three sizes, the largest size (L), being sand that will pass a sieve of 5 meshes per lineal inch, but will not pass a sieve of 15 meshes per inch; the medium size, (M), being sand that will pass a 15 mesh sieve, but will not pass a 50 mesh sieve; and the fine size, (F), being sand that will pass a 50 mesh sieve. It will be found that if the three sizes be mixed the densest mixture with the least voids will be obtained when 4 parts of the large size, no parts medium and 4 parts fine size are used, i. e., L6, Mo, F4 mixture. The weight of a cubic yard of either coarse, medium or fine grade, will be 2,190 pounds, if the specific gravity of sand be taken at 2.65. For this weight the voids will be 51 per cent. If, however, the densest mixture obtained by using 6 parts of L, o parts M and 4 parts of F sands be weighed, it will be found to weigh 2,480 pounds per cubic yard. This is equivalent to 36 per cent. voids. Thus we see that a correct grading of fine and coarse grains is necessary to obtain densest mixture.

The shape of the grains has a pronounced effect upon the percentage of voids, rounded grains having less voids than angular grains. It was found by Feret that a mixture of L5, M3, F2 measured in a quart measure gave the following percentage of voids:

	Voids	
	Unshaken.	Shaken.
Natural sand, rounded grains	35.9%	25.6%
Crushed quartzite, anular grains	42.1%	27.4%
Crushed shells, flat grains	44.3%	31.8%
Residue of quartzite, flat grains	47.5%	34.6%

The measure was shaken until no further settlement could be produced.

The following test was made on sand by Mr. William B. Fuller: A dry sand having 34 per cent. void, shrank 9.6 per cent. in volume until it had 27 per cent. voids. The same sand moistened with 6 per cent. of water, and loose, had 44 per cent. voids, which were reduced to 31 per cent. by ramming. The same sand saturated with water had 33 per cent. voids and by thorough ramming its volume was reduced 8.5 per cent. until the sand had only 26.5 per cent. voids.

VOIDS IN SAND.

Locality.	Authority.	Voids.	Remarks.
Ohio River	C. E. Sherman	31%	Washed
Sandusky, O	W. H. Hall	40%	Lake
Franklin Co., O	C. E. Sherman	40%	Bank
Sandusky Bay, O	S. B. Newberry	32.3%	———
St. Louis, Mo	H. H. Henby	34.3%	Miss. River
Sault Ste. Marie	H. von Schon	41.7%	River
Chicago, Ill	H. P. Bordman	34 to 40%	———
Philadelphia, Pa	———	39%	Delaware River
Coast of Mass	———	31 to 34%	———
Boston, Mass	Geo. A. Kimball	33%	Clean
Cow Bay, L. I	Myron S. Falk	40.5%	———
Little Falls, N. J	W. B. Fuller	45.6%	———
Canton, Ill	G. W. Chandler	30%	Clean

The effect of the size of grains of the sand is shown in Table X.

TABLE X.—SIZES OF SAND GRAIN.

Held by a Sieve.	A	B	C	D
No. 10	35.3%			
No. 20	32.1%	12.8%	4.2%	11%
No. 30	14.6%	49.0%	12.5%	14%
No. 40			44.4%	
No. 50	9.6%	29.3%		53%
No. 100	4.9%	5.7%		
No. 200	2.0%	2.3%		
Voids	33%	39%	41.7%	31%

A is a fine gravel (containing 8% clay) used at Philadelphia.
B is a Delaware River sand.
C is a St. Mary's River sand.
D is a Green River, Ky., sand, clean and sharp.

Voids in Broken Stone and Gravel.—The percentage of voids in loose broken stone will depend upon the size of the stone and probably to some extent upon the character of stone used; as with sand the amount of voids will vary according to the sizes of the particles of stone or gravel—thus if uniformly graded from the largest size to crusher dust, the voids will be less than if the stone has been screened into uniform sizes. The densest mixture may be obtained by screening the stone and mixing the proper proportions of each size to secure a minimum of voids. In many cases it will be found that stone crusher run will give as dense a concrete as is desired without the additional expense of screening and mixing.

Pure quartz weighs about 165 pounds per cubic foot, hence broken quartz having 40 per cent. voids weighs 165 × 60% or 99 pounds per cubic foot.

Gravels are seldom composed entirely of quartz, but are usually made up also of stone like trap rock having a greater specific gravity, or of shales and sandstone, which have a lower specific gravity. When the specific gravity of a given gravel or stone is known the percentage of voids may be determined from the weight of a cubic foot of the loose materials. The specific gravities of different minerals and rocks are given in Tables XI. and XII, the percentages of voids may be obtained from Table XIII, while Table XIV gives the voids for various kinds of stone, according to different authorities.

TABLE XI.—SPECIFIC GRAVITY OF STONE.

(Condensed from Merrill's "Stones for Building.")

Trap, Boston, Mass.	2.78
" Duluth, Minn	2.8 to 3.0
" Jersey City, N. J.	3.03
" Staten Island, N. Y.	2.86
Gneiss, Madison Ave., N. Y.	2.92
Granite, New London, Conn.	2.66
" Greenwich, Conn.	2.84
" Vinalhaven, Me.	2.66
" Quincy, Mass.	2.66
" Barre, Vt.	2.65
Limestone, Joliet, Ill.	2.56
Quincy, Ill.	2.51 to 2.57
Limestone (oolitic) Bedford Ind	2.25 to 2.45
" Marquette, Mich.	2.34
" Glens Falls, N. Y.	2.70
" Lake Champlain, N. Y.	2.75
Sandstone, Portland, Conn.	2.64
" Haverstraw, N. Y.	2.13
" Medina, N. Y.	2.41
" Potsdam, N. Y.	2.60
" (grit) Berea, O.	2.12

TABLE XII.—SPECIFIC GRAVITY OF COMMON MINERALS AND ROCKS.

Apatite	2.92—3.25
Basalt	3.01
Calçite, $CaCO^3$	2.5 —2.73
Cassiterite, SnO^2	6.4 —7.1
Cerrusite, $PbCo^3$	6.46—6.48
Chalcopyrite, $CuFeS^2$	4.1 —4.3
Coal, anthracite	1.3 —1.84
Coal, bituminous	1.2 —1.5
Diabase	2.6 —3.03
Diorite	2.92
Dolomite, $CaMg(CO^3)^2$	2.8 —2.9
Feldspar	2.44—2.78
Felsite	2.65
Galena, PbS	7.25—7.77
Garnet	3.15—4.31
Gneiss	2.62—2.92
Granite	2.55—2.86
Gypsum	2.3 —3.28
Halite (salt) NaCl	2.1 —2.56
Hematite, Fe^2O^3	4.5 —5.3
Hornblende	3.05—3.47
Limonite, $Fe^4O^3(OH)^6$	3.6 —4.0
Limestone	2.35—2.87
Magnetite, Fe^3O^4	4.9 —5.2
Marble	2.08—2.85
Mica	2.75—3.1
Mica Schist	2.5 —2.9
Olivine	3.33—3.5
Porphyry	2.5 —2.6
Pyrite, FeS^2	4.83—5.2
Quartz, SiO^2	2.5 —2.8
Quartzite	2.6 —2.7
Sandstone	2.0 —2.78
" Medina	2.4
" Ohio	2.2
" Slaty	1.82
Shale	2.4 —2.8
Slate	2.5 —2.88
Sphalerite, ZnS	3.9 —4.2
Stibnite, Sb^2S^3	4.5 —4.6
Syenite	2.27—2.65
Talc	2.56—2.8
Trap	2.6 —3.0

As a rule it will be found that the voids in gravel are seldom less than 30 per cent. or more than 45 per cent. If the pebbles vary considerably in size, the voids will approximate the lower percentage, but if they are of nearly uniform size the voids will approximate the higher percentage.

Gravel compacts more readily than broken stone, on account of the rounded shape of the pebbles, while the angular shape of the particles of broken stone prevent easy packing. It is stated by Prof. S. B. Newberry that the voids in Sandusky Bay gravel from 1/8 to 1/4 in. in size are 42.4 per cent., and for sizes from 1-20 to 1/4 in. are 35.9 per cent.

For stone passing a 2½ in. screen with the dust screened out, composed of mixture of Green River, Ky., bluestone and Ohio River washed gravel, Mr. William M. Hall gives the following voids:

Stone.		Gravel.	Voids in Mixture.
100%	with	0%	48%
80%	"	20%	44%
70%	"	30%	41%
60%	"	40%	38.5%
50%	"	50%	36%
0%	"	100%	35%

Hudson River trap rock and gravel of the same sizes as just given, had the following voids:

Trap.		Gravel.	Voids in Mixture.
100%	with	0%	50%
60%	"	40%	38.5%
50%	"	50%	36%
0%	"	100%	35%

Size of Cement Barrels.—There is no uniform practice among engineers in regard to the standard of volume used in measuring cement when proportioning concrete. In Tables IV and V, given above, Mr. Thacher uses cement measured loose and gives 6.56 barrels of cement to the cubic yard, which is 4.12 cu. ft. per barrel. The actual cubical contents of cement barrels used by cement manufacturers vary from 3.2 to 3.8 cu. ft.; 3.5 cu. ft. per barrel may be taken as a fair average. Correspondence with a number of cement manufacturers shows that the amount of cement measured loose, which is packed in the barrels, varies

TABLE XIII.

Specific gravity.	Weight in lbs. per cu. ft.	Weight in lbs. per cu. yd.	Weight in Pounds per Cubic Yard when Voids are—				
			30%	35%	40%	45%	50%
1.0	62.355	1,684	1,178	1,094	1,010	926	842
2.0	124.7	3,367	2,357	2,187	2,020	1,852	1,684
2.1	130.9	3,536	2,475	2,298	2,121	1,945	1,768
2.2	137.2	3,704	2,593	2,408	2,222	2,037	1,852
2.3	143.4	3,872	2,711	2,517	2,323	2,130	1,936
2.4	149.7	4,041	2,828	2,626	2,424	2,222	2,020
2.5	155.9	4,209	2,946	2,736	2,525	2,315	2,105
2.6	162.1	4,377	3,064	2,845	2,626	2,408	2,189
2.7	168.4	4,546	3,182	2,955	2,727	2,500	2,273
2.8	174.6	4,714	3,300	3,064	2,828	2,593	2,357
2.9	180.9	4,882	3,418	3,174	2,929	2,685	2,441
3.0	187.1	5,051	3,536	3,283	3,030	2,778	2,526
3.1	193.3	5,210	3,653	3,392	3,131	2,871	2,609
3.2	199.5	5,388	3,771	3,502	3,232	2,963	2,694
3.3	205.8	5,556	3,889	3,611	3,333	3,056	2,778
3.4	212.0	5,724	4,007	3,721	3,434	3,148	2,862
3.5	218.3	5,893	4,125	3,830	3,535	3,241	2,947

TABLE XIV.—VOIDS IN LOOSE BROKEN STONE.

Authority	% Voids.	Remarks.
Sabin	49.0	Limestone, crusher run after screening out ⅛-in. and under.
"	44.0	Limestone (1 part screenings mixed with 6 parts broken stone.)
Wm. M. Black	46.5	Screened and washed, 2 ins. and under.
J. J. R. Croes	47.5	Gneiss, after screening out ¼-in. and under.
S. B. Newberry	47.0	Chiefly about egg size.
H. P. Boardman	39 to 42	Chicago limestone, crusher run.
"	48 to 52	Chicago limestone, screened into sizes.
Wm. H. Hall	48.0	Green River limestone, 2½ ins. and smaller, dust screened out.
"	50.0	Hudson River trap, 2½ in. and smaller, dust screened out.
Wm. B. Fuller	47.6	New Jersey trap, crusher run, $^1/_6$ to 2.1 ins.
Geo. A. Kimball	49.5	Roxbury conglomerate, ½ to 2½ ins.
Myron S. Falk	48.0	Limestone, ½ to 3 ins.
W. H. Henby	43.0	" 2-in. size.
"	46.0	" 1½-in. size.
Feret	53.4	Stone, 1.6 to 2.4 ins.
"	51.7	" 0.8 to 1.6 in.
"	52.1	" 0.4 to 0.8 in.
A. W. Dow	45.3	Bluestone, 89% being 1½ to 2½ ins.
"	45.3	" 90% being $^1/_4$ to 1½ ins.
Taylor & Thompson	54.5	Trap, hard, 1 to 2½ ins.
"	54.5	" " ½ to 1 in.
"	45.0	" " 0 to 2½ ins.
"	51.2	" soft, ¾ to 2 ins.
G. W. Chandler	40.0	Canton, Ill.
Emile Low	39.0	Buffalo limestone, crusher run, dust in.
C. M. Saville	46.0	Crushed cobblestone, screened into sizes.

from 4 to 5 cu. ft., being usually from 4.2 to 4.5 cu. ft., depending upon its specific gravity, the method of measuring when loose and the density with which it is packed. The manufacturers agree, however, upon the amount of cement by weight per barrel, 380 lbs. net being taken as the standard. The American Society for Testing Materials recommends a weight of 94 lbs. per sack and 4 sacks or 376 lbs. net per barrel.

It will be well to assume each barrel of 380 lbs. to consist of 3.5 cu. ft., and use this as a basis for computing the quantities of sand and stone to be used. For a 1 : 2 : 4 mixture we will then have 1 barrel=3.5 cu. ft. of cement, 7 cu. ft. of sand and 14 cu. ft. of stone. If the sack be taken as the unit we have: 1 sack = 0.9 cu. ft. cement, 1.8 cu. ft. sand and 3.6 cu. ft. stone. Multiples of these quantities may be used when batches of greater or less size are desired.

For a 1:4:6 mixture the quantities are 1 barrel = 3.5 cu. ft. cement, 10.5 cu. ft. sand and 21 cu. ft. stone.

Such a method when the sand and stone are carefully measured will give a definite proportion of cement for the sand and stone, whereas if a volume of loose cement be the unit, no definite proportions can be assured unless a definite volume be arbitrarily assumed for the loose volume of a barrel of cement.

The matter of standard of measurement may be considered from another standpoint. When, in specifying the proportions, the volumes of the aggregate are not distinctly stated in terms of cubic feet of each material to a barrel of cement or the volume in cubic feet of a barrel of cement is not specified, undoubtedly

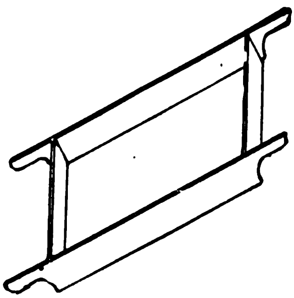


Fig. 20.—Measuring Box for Sand and Stone.

the contractor has the legal right to base the volumes of aggregate on the volume of a barrel of cement measured loose. Tables IV. and V., pages 54, 55, are based on cement in loose volume, it being assumed that 1 bbl. of cement will when loose measure 4.12 cu. ft.

The simplest method of measuring the aggregate is by the use of a bottomless box 8 or 10 ins. high, and of proper lineal dimensions to give the desired measuring volume. Thus if the volume of 1 barrel or 4 bags of cement be taken as 3.5 cu. ft. and used as the unit of measure and a 1 : 2 : 4 concrete is to be employed, a box (Fig. 20) 8 ins. deep and 2 ft. 7½ ins. by 4 ft. may be used for measuring the sand and stone. This box should be filled once with sand and twice with broken stone, care being taken to strike it off level and in no case permit the materials to be heaped up in any way.

Amount of Water.—There is considerable difference of opinion among engineers as to the quantity of water that should be used for mixing concrete. It was formerly supposed that a dry mixture gives a much stronger concrete than a medium or wet mixture, but this is by no means certain, and, on account of practical considerations, wet mixtures are much more satisfactory for concretes used with reinforcement. Myron S. Falk states, in his "Cements, Mortars and Concretes," that tests made by J. W. Sussex in thesis work at the University of Illinois in 1903, to determine the relative strength of wet and dry mixtures, show the greatest strength for wet mixtures (3 months old). The tests were made on forty-five 6-in. cubes, mixed with three different percentages of water, and were broken at the ages of seven days, one month and three months. The concrete was composed of one volume of Portland cement, three volumes of sand containing a small percentage of fine gravel, and six volumes of crushed limestone. The tests were made with the three degrees of plasticity noted in Table XV, and also two degrees of tamping, light and hard. Each result shown is the average of three tests. At the end of three months the wet concretes furnished the greatest ultimate strength, although at the end of seven days and one month the medium specimens furnished the highest ultimate strength, whether tamped lightly or hard.

TABLE XV.

	Crushing Strength in Lbs. per Sq. Inch.				
	Dry		Medium		
Age.	Lightly Tamped.	Heavily Tamped.	Lightly Tamped.	Heavily Tamped.	Wet.
7 days	1,200	1,340	2,280	1,330	1,040
1 month	1,750	1,960	2,290	2,560	2,230
3 months	2,500	2,600	2,150	2,590	3,040

Mr. George W. Rafter has recorded in the report of the State Engineer of New York, for 1897, a series of tests showing the relative strength of concrete mixed with varying percentages of water. Mr. Rafter does not express the ingredients of a concrete in the usual way. In order to make his tests comparable with others they have been reduced to the usual form in Table XVI.

As will be seen, in most cases the dry mixtures have a slightly greater strength than the wet, while little uniformity is shown in the strength of the plastic mixtures.

Under ordinary conditions, a more dense and homogeneous concrete will be secured by using a wet mixture than a dry mixture. Then, too, there is much more assurance that the reinforcement will be thoroughly embedded if the concrete is used wet enough to flow readily with moderate ramming. This is especially desirable if the reinforcing units are small or are placed closely together. Again, where a dry mixture is used much ramming is necessary to secure good results; this is at times

TABLE XVI.

	Ultimate Crushing Strength, in Lbs., per Sq. In. Average of 4 Specimens.		
Proportion.	Excess of Water.	Plastic.	Dry.
1:1:4	3,256	4,123	3,966
1:1:5	3,764	4,072	4,267
1:2:6	3.168	2,960	3.404
1:2:7	2,847	2,777	2,888
1:3:8	2,016	2,027	2,179
1:3:9½	1,723	2,056	2,207
1:4:10½	1,670	1,750	1,671
1:4:12	1,767	1,600	1,810
1:5:12	1,400	1,465	1,559
:5:15	1,441	1,586	1,537

difficult to do, is expensive, and, on account of confined spaces, is at times impossible. Water is cheaper than ramming, and if used with judgment will give practically the same results. Also there is less danger of displacing the reinforcement. In some cases where pipes are formed by running grout of quick setting cement into moulds, ramming is entirely dispensed with. Until recently, M. Hennebique used dry mixtures, but he now uses more water with less ramming.

CHAPTER V.

MIXING CONCRETE.

Concrete may be mixed by hand or by machinery. If the work is thoroughly and conscientiously done, a good concrete can be secured by either method. If the job is a small one, in almost every case hand mixing will be found to be the cheaper, although if a mixer is available, it should be used, as by its use there is less liability of securing a poor mix. The cost of installing a plant, by which economy in mixing may be secured, will more than balance the saving obtained by machine mixing, unless considerable concrete is to be used.

Whichever method is used, the inspection should be careful and continuous. The points to be insisted upon are: (1) exact measurement of materials; (2) thorough mixing until the color and consistency of the mass are uniform throughout; (3) that the correct amount of water is used; and (4) that proper care is taken in dumping the concrete in place.

Long Time Mixing.—The strength of a concrete will depend largely upon the thoroughness with which the surface of each and every particle of the sand and broken stone is coated with a film of cement mortar. Of course the efficiency of the machine or the thoroughness with which the men do their work will govern to a large extent the thoroughness of mixing, but other things being equal the strength of the concrete will vary with the time of mixing.

A series of tests was made by Mr. Clarence Coleman, M. Am. Soc. C. E., to determine the effect of mixing for different periods of time. The tests were made on 1 cement to 3 sand briquettes. The mixing was by hand, in a cast-iron box with inclined ends, using a hoe almost as wide as the box. As shown in Table XVII, the briquettes were broken at ages varying from 7 days to 2 years.

Each batch for making five briquettes was mixed from one to ten minutes, advancing one minute for each batch. As will be seen, the gain in strength from 1 to 10 minutes was 54, 42 and

20.7 per cent. for 1 month, 3 months and 2 years respectively. Hence we see the desirability of thorough mixing.

TABLE XVII.

EFFECT OF TIME OF MIXING ON STRENGTH OF CEMENT MORTAR (1 CEMENT TO 3 SAND).

Time of mixing.	Water per cent of dry ingredients.	Mean Tensile Strength of Briquettes After —Days— 7, lbs.	28, lbs.	—Months— 3, lbs.	6, lbs.	—Years— 1, lbs.	2, lbs.
1 minute	8.25	231.6	317	397	437	435	429
2 minutes	8.25	274.4	366	425	447	468	430
3 "	8.25	288.2	396	454	516	521	459
4 "	8.25	306.8	418	466	534	536	490
5 "	8.25	324.6	436	495	546	532	515
6 "	8.25	335.0	446	528	554	559	481
7 "	8.37	344.8	446	500	509	585	530
8 "	8.50	387.2	471	530	571	563	511
9 "	8.75	362.2	469	538	603	601	530
10 "	8.87	368.6	488	564	612	615	524

Machine vs. Hand Mixing.—Tests made by U. S. Government engineers at Duluth, Minn.,* to determine the relative strength of concrete mixed by hand and by machine (a cube mixer) showed that at 7 days hand mixed concrete only possessed 53 per cent. of the strength of machine mixed concrete, at 28 days 77 per cent., at 6 months 84 per cent. and at 1 year 88 per cent.

Details of these tests were as follows: Concrete was composed of 1 part Portland cement, 10.18 parts aggregates.

TENSILE TESTS.

Age 7 days.	High.	Low.	Average.
Machine mixed sample	260	243	253
Hand mixed sample	159	113	134
Hand mixture had 53 per cent of strength of machine sample.			
Age 28 days.	High.	Low.	Average.
Machine mixed sample	294	249	274
Hand mixed sample	231	197	211
Hand mixed sample had 77 per cent of strength of machine sample.			
Age 6 months.	High.	Low.	Average.
Machine mixed sample	441	345	388
Hand mixed sample	355	298	324
Hand mixed sample had 84 per cent of strength of machine sample.			
Age 1 year.	High.	Low.	Average.
Machine mixed sample	435	367	391
Hand mixed sample	369	312	343
Hand mixed sample had 88 per cent of the strength of machine mixed sample at end of one year.			

It should be noted in this connection that variation in strength from highest to lowest was greatest for the hand mixed samples.

*Report Chief of Engineers, U. S. Army, 1904, p. 3,795.

Thus more uniform results are to be expected when the mixing is done by machinery.

Hand Mixing.—For hand mixing the platform should be located as near as possible to the work, and so situated that the cement, sand and stone can be dumped conveniently near to it. Conditions should be such that the mixed product can be easily removed and deposited. The platform should be from 12 to 20 ft. square, and constructed of 2-in. plank dressed on one side and nailed to 2x4-in. stringers, 4 or 5 ft. apart. In many cases it will be found that a 2 or 3-in. strip nailed around the edge of the platform will prevent wasting the material. A convenient method of measuring the materials has been given on page 66.

Different methods of hand mixing are employed by engineers. The writer prefers to mix the cement and sand dry on a platform or mixing board, the mixing being continued until the dry material is of uniform color. Water is then added, and the mixture turned over until a cement mortar of uniform consistency is obtained. The stone, having been previously thoroughly wet, is then added and the whole mass turned over until a satisfactory mixture is obtained.

Another method is to mix the cement, sand and stone dry; then add water, slowly turning over, and mixing as the water is added. It is customary to spread the materials evenly over the board, and add water slowly as the process proceeds. Four men take their positions, two on each side of the material, two shoveling left and two right handed. The material is then shoveled towards the ends of the board, care being taken to turn the shovel completely over each time. The material at the next turn is shoveled towards the centre of the board. Three or more turns are necessary to obtain a good mixture.

Machine Mixers.—Machine mixers used in this country may be grouped into three classes: (1) Continuous, (2) batch and (3) gravity mixers.

The continuous mixer usually consists of a central shaft, to which are attached arms or paddles, rotating in a long trough. The paddles are set at such an angle that they have an endless screw action, cutting and pushing the materials down toward the lower end of the trough, out of which a continuous stream of concrete flows. Two types of machines are used: Figure 21 shows that in which the materials are thrown into the trough

Fig. 21.—Plain Trough Mixer.

by men standing at the sides of the machine, part shoveling sand and part stone. With this machine the uniformity of the product depends upon the ability of the men to shovel evenly, a most uncertain quality. In the other type of machine, shown in Fig. 22, the materials are placed in hoppers at the head of the

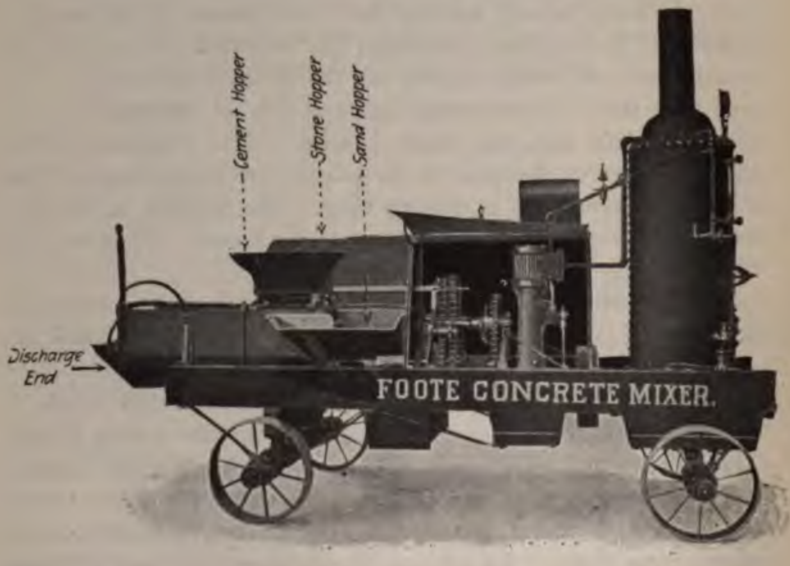


Fig. 22.—Trough Mixer with Automatic Measuring Device.

trough and are automatically measured. The figure shows the arrangement of the cement, sand and stone hoppers. It also shows the operating engine and the various shafts leading to the feed screws in the several hoppers. With this machine a more uniform mixture is obtained. It has been the author's experience, however, that continuous mixers should be avoided for reinforced concrete work. Many specifications prohibit the use of continuous mixers.

Batch mixers may be classified as follows: (1) The cube mixer, (2) the double cone or Smith mixer, and (3) the Ransome or drum mixer, which does not tilt. Many other mixers are

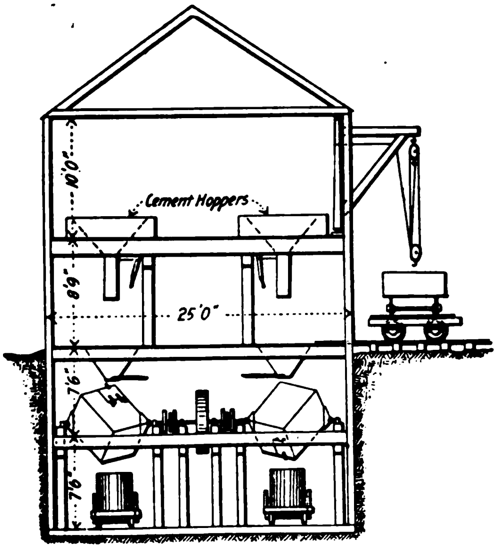


Fig. 22.—Twin Cube Mixer Plant.

on the market, but all closely resemble some one of the above named types. In operation the batch mixer is turned 10 or 15 times or more after charging and then the materials are discharged.

Cube Mixer.—One of the oldest forms of mixers is the cube mixer (Figure 23), which consists simply of a cubical steel box mounted on trunnions, or a shaft passing through diagonally opposite corners. The materials are put in through a door in the side of the box; the door is closed, and the mixer revolved about the trunnions as an axis; the door is then opened and the concrete dumped out. Figure 24 shows the improved Chicago cube

Fig. 24.—Improved Chicago Cube Mixer.

mixer. In this machine the materials are introduced at one corner and flow out at the other.

Smith Mixer.—The Smith, or double cone, mixer is shown in Figure 25, and consists of a revolving drum having the shape of a double cone. This machine revolves on its horizontal axis.

Fig. 25.—Smith Double Cone Mixer.

Baffle plates, or deflecting wings, are attached to the inside of the shell; these, as it revolves, lift up and turn over the materials, thoroughly mixing them. The drum is open at both ends. The materials are introduced at one end, and, after the mixing is completed, are discharged at the other end by tilting the machine from its horizontal position so that the materials will run out. The concrete is visible throughout the whole operation, and can be discharged without stopping the machine.

Ransome Mixer.—The Ransome, or drum, mixer (Fig. 26)

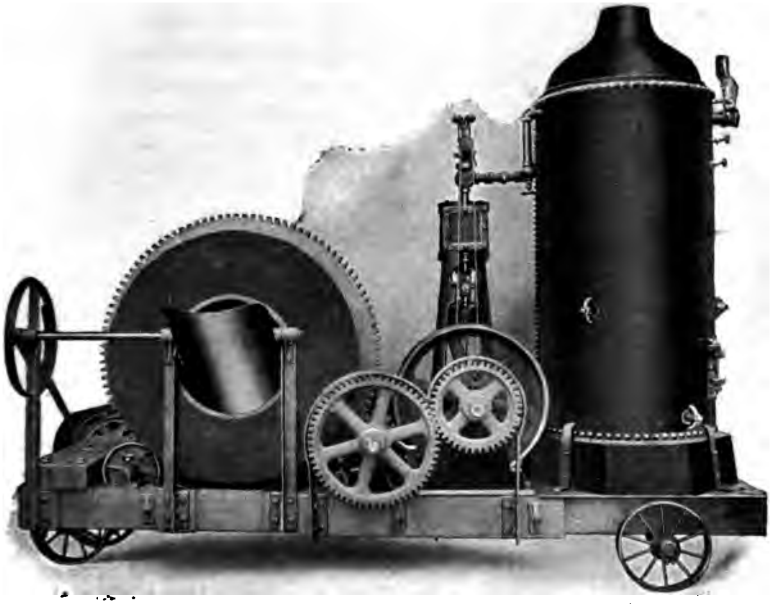


Fig. 26.—Ransome Drum Mixer.

has a cylindrical drum revolving about its horizontal axis. The machine rests upon friction rollers, and is driven by cog wheels working in the cogged rim attached to the circumference of the drum. The whole is driven by steam, compressed air, electricity or by a gas engine. Inside the drum and riveted to the circumference of the shell are steel scoops by which the concrete is lifted and turned over. The scoops are staggered in such a way as to give the materials a back and forth travel in the drum. Passing through the drum and supported upon the truck is a discharge chute, which can be tilted so as to receive the concrete as it drops from the scoops and deliver it into buckets, cars or wheelbarrows used to carry it to the work.

Gravity Mixer.—In one type of gravity mixer the falling materials strike baffle plates, which throw them together in their descent through the machine. Figure 27 shows the Gilbreth gravity mixer. It consists of a funnel-shaped receiving end attached to a box-shaped chute having deflector plates attached to the sides and set at an angle of 20 or 25 degrees to the vertical, together with rows of pins or rods set at frequent intervals along the length of the chute. The baffle plates, or deflectors, throw the material from side to side, and the pins mix it as it descends. The materials are measured in layers on a platform above the machine, and then shoveled into it. The principal objection to this machine seems to be the difficulty of delivering the proper amount of each material to the machine simultaneously, so as to secure a perfectly uniform mixture.

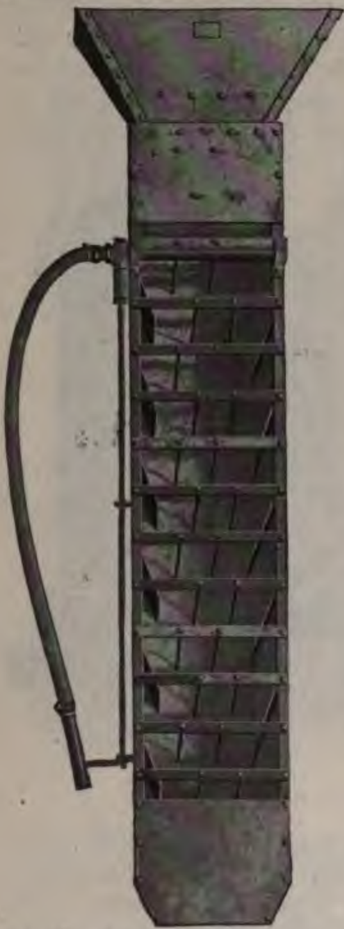
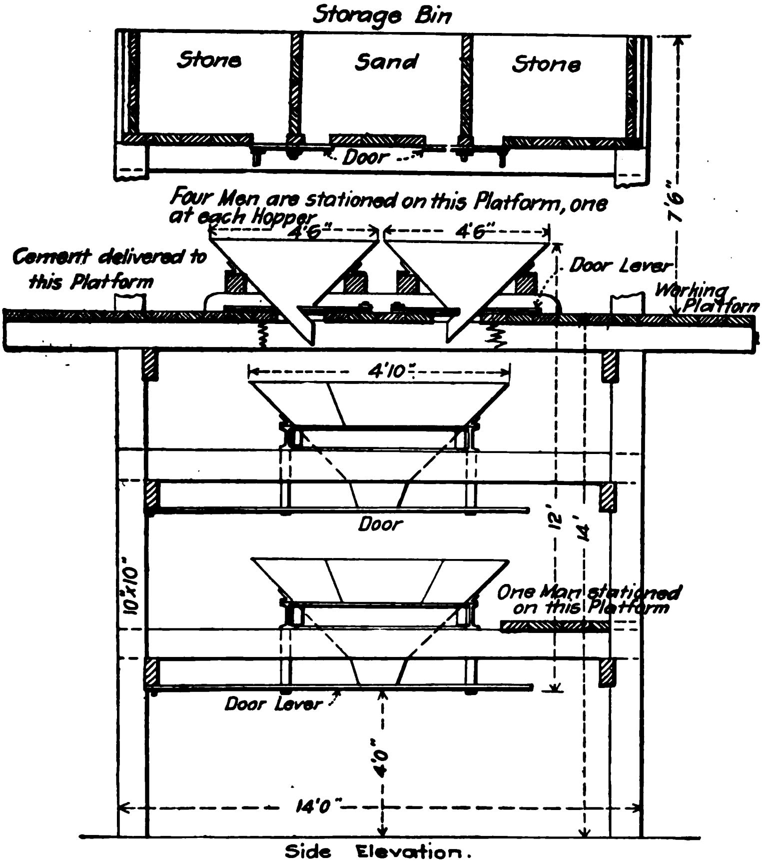


Fig. 27.—Gilbreth Gravity Mixer.

The Hains gravity mixer (Fig. 28) consists of a series of hoppers placed one above the other. The materials are dropped from one hopper to another, water being added, and are supposed to turn over and mix in their descent. This mixer has been used by government engineers on work of great magnitude, and it is stated that a very good concrete was secured.

The author believes that gravity mixers should not be used on reinforced concrete work where it is almost absolutely essential that a perfectly uniform mix shall be secured.

Automatic Measurers.—A number of machines for measuring the materials automatically have been devised, but unless a large quantity of concrete is used the expense of their installation will not be warranted. Figure 29 shows one type of automatic measurer known as the Trump measurer. This machine consists of several bottomless storage cylinders, from beneath which the



Side Elevation.
Fig. 28.—Hains' Gravity Mixer.

materials flow out on to a revolving platform and are scraped off by stationary arms resting upon the top of the platform and projecting into each material a sufficient distance to scrape off the proper amount of each. Figure 30 shows the Treadwell measurer, which consists of several drums, one for each material, placed directly under bins containing cement, sand and stone, and rotating upon the same horizontal shaft. The quantity of material is regulated by means of gates in the bins and the speed of rotation of the drums.

Automatic weighing machines are at times used when the proportioning is done by weight. These consist of a series of automatic tipping buckets placed under spouts leading from the storage bins. When the proper weight of material is in the bucket it automatically tips, shuts a valve in the spout and empties into the hopper leading to the machine. When all three materials have reached the hopper a valve opens and they are emptied into the mixing machine. When different sizes of stone are used to secure a well balanced aggregate bins and automatic tipping buckets are supplied for each size.

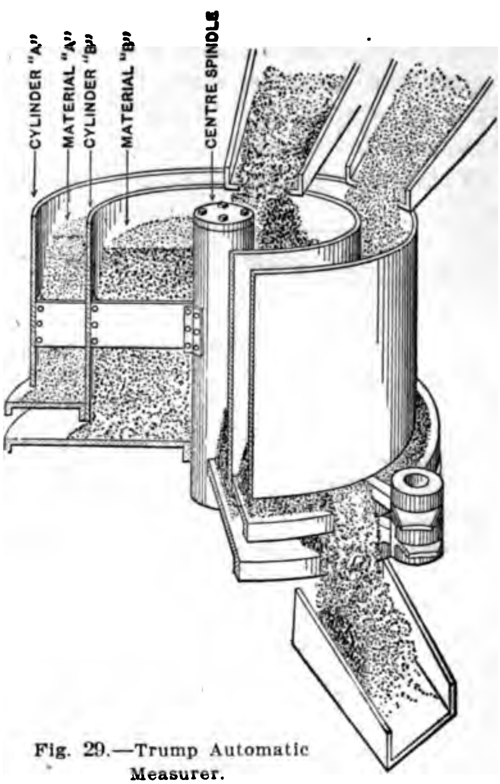


Fig. 29.—Trump Automatic Measurer.

Mixing Plants.—Great economy may often be effected by carefully designing the plant for handling both the raw materials and the concrete. Study should be made of local conditions, and the layout of the plant should be sketched on paper. Designs should be made of several arrangements of the machinery, together with estimates of the first cost of the plant and the probable expense of operation. Local conditions will largely affect the design and determine the layout of the plant.

When the quantity of materials to be handled is large, the introduction of conveying machinery will often effect a great saving. Both the aggregate and the mixed product are handled by this machinery. One of the simplest and most effective methods of delivering concrete to the upper stories of a reinforced concrete building which the author has seen was by means of a "mechanical hod-carrier." This apparatus consists of a light tower with an endless double chain working over sheaves at the top and bottom and driven by the engine operating the mixer. Hooks are attached to the hoisting chains, the concrete is delivered by the mixing machine into buckets, a man hangs the bucket on the hooks attached to the hoisting

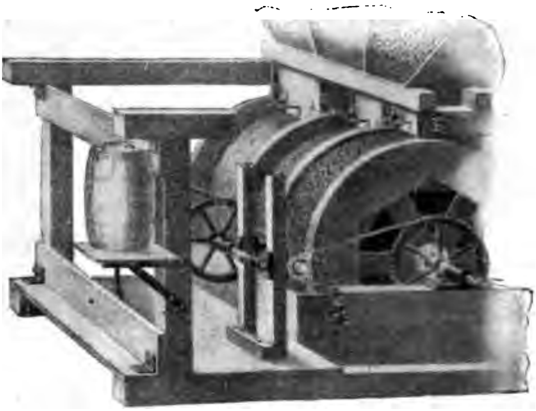


Fig. 30.—Treadwell Automatic Measurer.

chain, and another man removes them when they reach the story where the concreting is being done. It has been found that this machine hoists the concrete as fast as it can be conveniently put in place.

The raw materials should be delivered as near as possible to the machine. At times they are unloaded on platforms or into bins placed above the machine. Chutes are provided from the bins for conveying the materials to the machine or they are shoveled directly into it after being measured. Bucket elevators may often be used with economy to raise the materials to the platform or bins above the machine.

One of the largest cost items in mixing concrete is that of shoveling materials into the machine. The hoisting bucket shown in Fig. 24 is used for raising the materials to the hopper of the mixing machine. The materials may be dumped directly into the bucket from wheelbarrows or wagons, and then raised by a power hoist operated by the engine used to drive the mixer. Various modifications of this power hoist are used on different types of machines.

The mixing plant shown in Fig. 23 was used by United States engineers on the construction of a canal at the cascades of the Columbia River, Oregon. The capacity of the plant was 250 cubic yards per day. It is stated that the cost of mixing and placing by a chute was $0.434 per cubic yard.

The mixing plant shown in Figure 31 was used in the con-

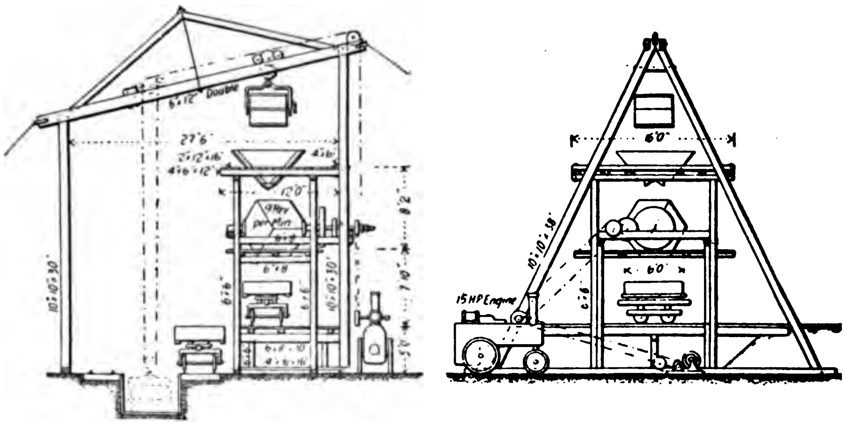


Fig 31.—Mixing Plant, Illinois & Mississippi Canal Lock Work.

struction of the concrete locks on the Illinois and Mississippi Canal. The track for the trolley from which the hoisting bucket was hung was the lower chord of a King post truss. The truss was supported by two A-frames, one having legs 30 ft. long and the other having legs 38 ft. long. A pit was dug under the truss and tracks laid on each side of the pit so that dump cars could readily deliver material into a charging box placed in the pit. The charging box was 3 ft. 8 ins. square and 3 ft. deep, holding 40 cubic feet, and was raised by a ½-in. steel cable running through a pair of double blocks. The slope of the lower chord of the truss was such that the cable hoisted the bucket and carried it along the truss without the use of latching devices. This, it should be noticed, is a very simple and ingenious hoisting and conveying apparatus.

A 15 H. P. portable engine operated the hoist with one pulley, and its other pulley operated the friction clutch driving the 4-ft. cubical concrete mixer. Above the mixer was a hopper, and under the mixer the track for the dump cars that carried the concrete to the lock walls. It was found necessary to lower the hopper 6 ins. lower than shown to prevent spilling the materials. It was also found desirable to reduce the distance between the mixer and the lower platform by 9 ins.

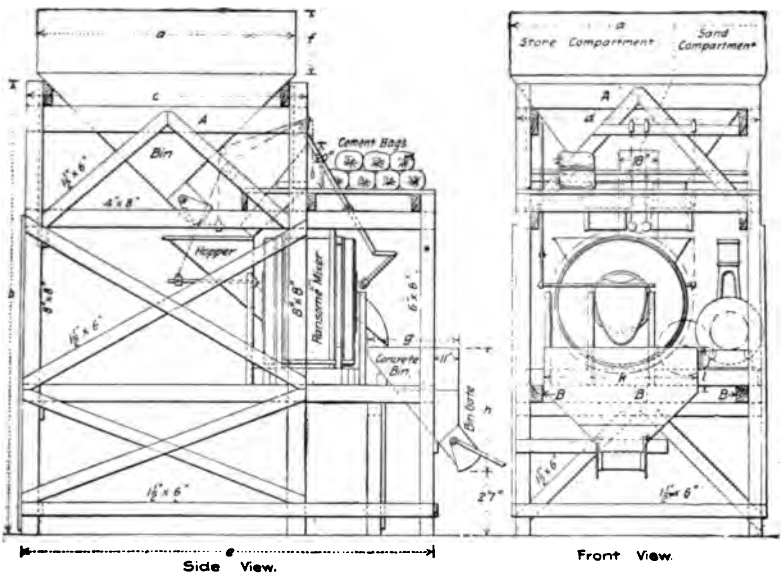


Fig. 32.—Standard Ransome Mixing Plant.

and to place diagonal timber braces under the timbers supporting the axis of the mixer. Nine revolutions of the mixer secured a perfect mixture of the concrete. The belt hoist, trolley, charging box and cubical mixer, with the necessary shafting, gearing, etc., cost $706, delivered, and the timber, framing and erection cost $300 more.

A mixing plant designed for use with a Ransome mixer is shown in Fig. 32. The bins for holding the stone and sand are supported by a scaffolding above a hopper in which the

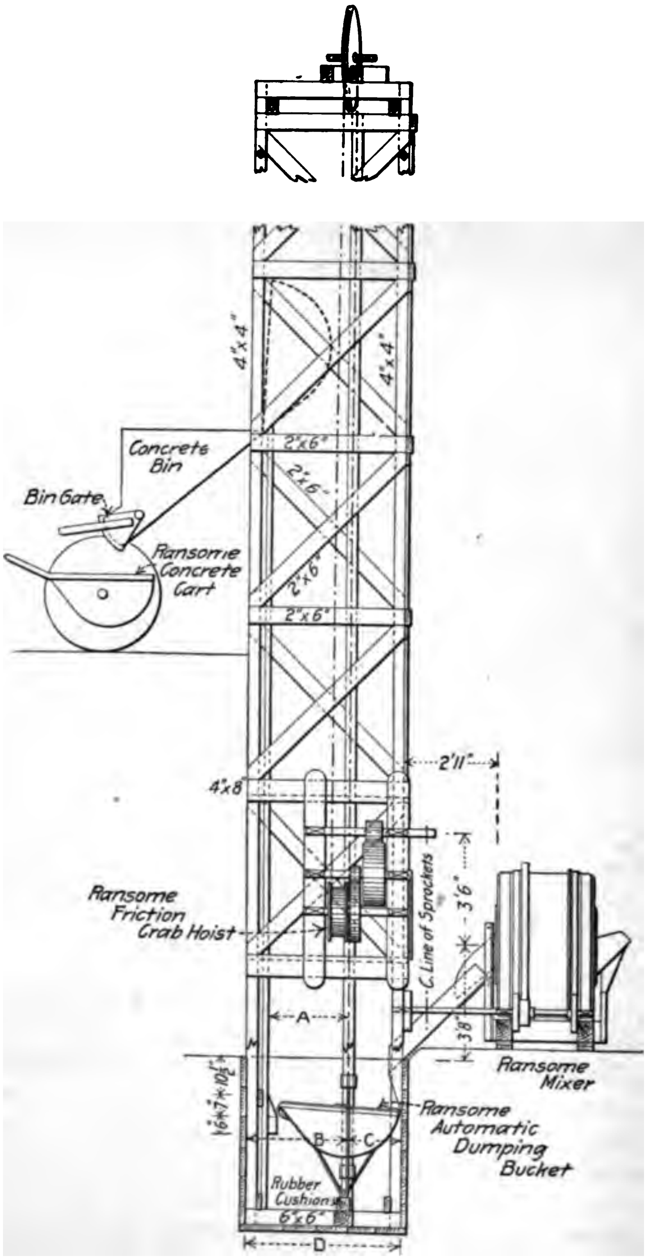


Fig. 33.—Ransome Hoist for Concrete.

proper proportions for each mix are deposited and then dumped into the mixer. After the batch has been properly mixed it is dumped into a concrete bin, from which it is drawn as needed. The proper dimensions for the various parts of the plant are shown in the following table. The numerals refer to the different sizes of Ransome mixers. The capacity per hour

Key Letter	No. of Mixer 2	No. of Mixer 8	No. of Mixer 4	Key Letter	No. of Mixer 2	No. of Mixer 8	No. of Mixer 4
a	10'-0"	11'- 0"	12'- 0"	g	3'-6"	4'- 0"	4'- 6"
b	17'-9"	18'-11"	20'- 0"	h	4'-9"	5'- 4"	5'- 8"
c	10'-7"	11'- 7"	12'- 7"	k	6'-0"	7'- 0"	7'- 6"
d	9'-6"	10'- 6"	11'- 6"	l	1'-9"	1'-10"	1'-11"
e	15'-9"	17'-10"	19'- 9"	Beams A	4'x12"	3'x14"	4'x14"
f	2'-6"	2'- 0"	2'- 3"	Beams B	6'x 8"	6'x 0"	6'x10"

of each of these mixers, respectively, is 20, 30 and 40 cubic yards. Any one of the well-known types of mixers may be used with this mixing plant.

The Ransome hoist may be used in connection with mixing plants, either for elevating the sand and stone or the concrete from the mixer to the various floors when used in building construction.

Figure 33 shows the arrangement of this hoist, which consists essentially of a framework supporting a sheave for the hoisting cable, and having guides for hoisting the bucket. When the bucket reaches the desired height it is self-dumping; the position is shown by the dotted lines in the annexed figure.

In the construction of the Bush Terminal Building in Brooklyn one of these hoists was used. The chief engineer stated to the author that the self-dumping arrangement did not prove satisfactory and it was necessary to use a crowbar for dumping the bucket. However, if the machinery is properly adjusted the self-dumping arrangement should prove satisfactory.

In Engineering-Contracting for February, 1906, was published the following description and costs obtained by using a mixing plant (Fig. 34) with a Ransome hoist to elevate materials to the bins. This was used in connection with a Ransome No. 4 mixer in the new portion of the terminal at the Grand Central Station of the New York Central R. R., New York City. The concrete was delivered from the mixer into two dump cars of the end-dump pattern, each holding 1 cubic yard. These cars run on a light track (2-ft. gauge). As

Fig. 34.—Concrete Mixing Plant, New York Central Terminal Work.

there was no room for a switch these cars were run on a single track and had to wait for each other. Four men were used to push each car, making 8 men transporting the concrete. Two more men were necessary to assist in dumping the cars and cleaning the track of any concrete which lodged upon it.

In the construction of a retaining wall 250 cubic yards of concrete were deposited in 8 hours. The cost of operation of this plant was as follows, wages being $1.50 per day for laborers:

	Per Day.
2 men carrying cement	$3.00
6 men shoveling sand	9.00
17 men shoveling stone	25.00
11 men wheeling stone	16.00
2 men at stone and sand bins	3.00
2 men opening cement bags	3.00
1 man dumping hopper	1.50
1 man dumping mixer	1.50
1 man chaining chute of mixer, etc	1.50
8 men pushing 2 carts	12.00
2 men cleaning track	3.00
7 men spading concrete	10.50
1 motorman or engineer	3.00
1 foreman	5.00
Electricity or steam power (estimated)	7.00
Total, 250 cu. yds., at 41.6 cts	$104.00

It will be noted that 6 men were employed in shoveling sand and 17 men in shoveling stone, while 11 men were wheeling stone, making a total cost for these items of $50.00. Where dump cars, such as Fig. 35, manufactured by Continental Car

Fig. 35.—Dump Car for Handling Concrete Materials.

& Equipment Co., or those of other well-known firms, can be used to deliver the materials at the site, unloading in bins having chutes to convey the sand and stone to the hoisting bucket, the major part of this portion of the cost can be eliminated. Thus we can see that if a plant is carefully designed, and a moderate outlay made for bins, tracks, etc., the cost of mixing can be reduced to a surprisingly low figure. Thus if the cost of this part of the handling be cut in two, which may easily be done by careful designing, the total cost of mixing will be $\$79 \div 250 = 31.6$ cents per cubic yard.

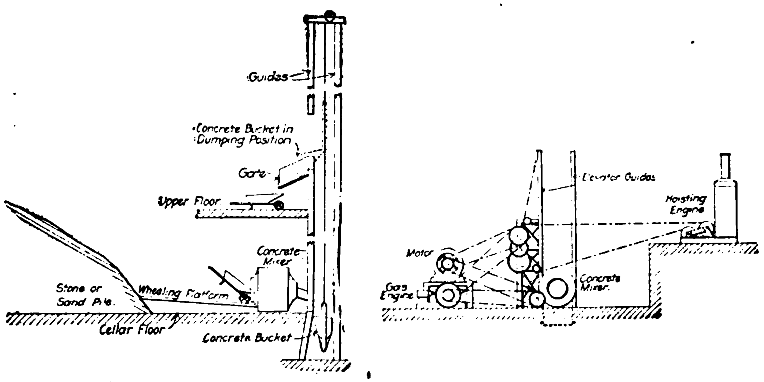


Fig. 36.—Concrete Mixing and Hoisting Plant, Ingalls Building, Cincinnati, Ohio.

The concrete mixing plant shown in Fig. 36 was used in the construction of the Ingalls Building. A large portion of the basement was utilized for the storage of sand, stone and cement. On one side was located the concrete mixer and gas engine used in driving same. Adjacent to this was the hoist used for elevating the concrete to the upper floors. The hoist consists of the usual guides, which act as a support for the sheave at the top, and a concrete bucket was used to raise the material, the concrete being dumped directly from the mixer into the bucket and then hoisted to the desired floor.

CHAPTER VI.

PLACING CONCRETE.

After the concrete has been thoroughly mixed it is usually conveyed to the work in wheelbarrows, dump cars running on a light tramway, or buckets handled by cranes, and deposited in 6 or 8 in. layers. Push carts may be used in place of wheelbarrows. In depositing concrete dumping from a height of several feet should be avoided where possible, as the stone and mortar may become more or less separated. After being deposited the concrete is thoroughly rammed until a film of water flushes slightly to the surface. Too severe ramming when the concrete is dry will injure it, by forcing the larger stone to the bottom, and, if the ramming is continued too long, the setting will be delayed and the strength permanently impaired. The amount of ramming which can be done will depend upon the amount of water used in mixing. Very wet concretes can not be rammed at all.

When possible a section of concrete work should be carried on continuously until completed, for, if the work is stopped, lines of cleavage are apt to be formed. When it is impossible to carry on the work continuously the points at which the work is stopped should be chosen so that any weakness which may develop will do the least possible harm. Before beginning concreting anew, the surface at the joint should be cleaned, roughened and washed with neat cement paste having a consistency of cream. A layer of concrete is then spread on, and the depositing continued as before. Every precaution should be taken to place the reinforcement in its proper position and retain it in place, to see that all voids are filled, and that the concrete after depositing is as homogeneous as possible.

The consistency of the mixture will often govern the methods of handling and depositing. With a dry mixture there is a tendency for some of the stone to separate from the mortar on the slightest provocation. A dry mixture may be described as having the consistency of damp earth. A medium mixture is soft and quakes like liver when rammed. A wet mixture is too soft to sustain a man, and so thin as to flow easily. Great care is necessary in handling and placing dry concrete to secure a uniform mixture throughout the structure. With a wet mixture there is less tendency for the materials to separate. When quite wet care must be taken that a part of the semi-fluid does not slop out of the wheelbarrows or buckets while being conveyed from the mixer to the work.

At times, when the mixer can be located conveniently near, the concrete may be run through troughs or chutes directly into place. Quite a wet, slushy consistency will prove the most satisfactory under these circumstances.

A wet, slushy concrete flows more readily down and around the reinforcement when the latter is placed in confined spaces. With wet concrete, tamping is not possible, but the concrete is usually cut with a spade, a wedged shaped tamping bar or a tamping bar with a thin narrow blade.

Care must be taken in placing the concrete in the narrow spaces of the forms, so that stone pockets will not be formed. In using wet concrete the forms should be so nearly water tight that the thin cement grout will not drain out, leaving voids and stone without a cement coating.

Grouting.—Grout is a thin or liquid cement mortar. The mortar may be neat or have various proportions of sand added, say from ½ to 2 parts to one part of cement. The neat cement, however, will prove the most satisfactory, as there is a tendency for the cement and sand to separate and form layers of sand and cement. Grout was formerly extensively used in the construction of bridge piers. It was customary, after laying the large backing stone in place, taking care in all cases to break joints in both directions, so as to bond the entire wall together both longitudinally and transversely, to fill the vacant spaces with broken stone of various sizes and then the grout work, i. e., pour liquid mortar into the open spaces until entirely filled. There is danger of not filling the spaces fully if the grout is too thick or the work not properly done, in which case a part of the masonry will not be much better than dry rubble.

Grouting is not now considered a first-class method of construction. It has, however, been used successfully in many cases, and will at times prove useful when, on account of local conditions, other methods cannot be used. It has been successfully used for subaqueous foundation work by English engineers, both in India and Europe.

The proper method of mixing grout is to mix the cement, on a flat platform or the ordinary mixing board, to the consistency of stiff paste, then place it in a tub and slightly thin it down by adding water in very small quantities and stirring until the paste is reduced to a thick grout, or just soft enough to leave the bucket. It should be poured as rapidly as possible and the pouring continued until completed. The faster the grout is poured and the more continuous the flow is kept up the better will be the results obtained.

Rubble Concrete.—In massive work large stones are sometimes embedded in concrete, forming what is known as rubble concrete. In such structures as dams, lock walls, breakwaters, retaining walls and, in many cases, bridge piers and abutments, a considerable reduction in cost may be obtained by the introduction of large stone without sacrificing in any way the strength or fitness of the structure. The stones thus embedded should be perfectly sound, and when large should not lie nearer one to another or the face of the wall than from three to six inches. The concrete should be mixed quite wet, and much care taken to completely surround each stone with a compact mass of concrete. The stones should be settled in the concrete already laid far enough to give them a full bed. The stone may be placed by hand or with a derrick.

Forms may be used as in ordinary concrete work, but at times the rubble concrete is laid as a backing to facing masonry, in which case the facing masonry is laid first and retains the rubble concrete until set. Masonry of this kind was used in the construction of the piers of the Poughkeepsie Bridge. A bed of concrete is usually put in place, and what is known as one or two man stones then put in position, care being taken to keep the stones from bearing directly one upon another, then another bed of concrete laid, and so on.

The quantity of rubble used will depend on the size and shape of the stone and the method and care with which they are placed. The percentage in different pieces of work of the character may vary from 20 per cent. to 65 per cent. of the whole mass.

About one-third of the total volume of the concrete used in the construction of a masonry drydock at the Charlestown Navy Yard was rubble stone. A dry mixed concrete was used. Stones averaging about one-half cubic yard in volume and having approximately square faces and level beds were placed about 18 ins. apart in all directions. A concrete composed of one part Portland cement, two parts sand and five parts gravel was deposited with buckets and thoroughly rammed, and the stones, after being thoroughly washed with a hose, were placed by derrick. If a stone was not properly bedded when dropped in place the derrick picked up a heavy weight and allowed it to drop several times upon the stone to ram it in place. It would appear that an equally good result at considerably less labor cost would have been obtained had a wet concrete been used without ramming.

In the construction of a dam on the Quinebaug River, in Connecticut, a rubble concrete was used. The dam had a height varying from 30 to 45 ft. above bedrock. The material composing the concrete consisted of bank sand and gravel excavated from bars in the bed of the river. The gravel was granitic in character, uniform in composition and in well graded sizes. A moist concrete was used. Care was taken to so place the large masses of rock that no voids or hollows would exist in the finished wall. The rocks and boulders were taken from the site of the dam foundation and were of varying sizes, limited by the size of the hoisting machinery. Stones as large as two to two and a half cubic yards of concrete were used in the construction of this dam. Above them other rocks were placed, gradually reducing the size toward the top of the dam. The concrete was used for bedding the stone in a manner similar to that in which ordinary mortar is used in building a rubble masonry wall. Some 12,000 cubic yards of concrete were used in the construction of this dam, and sometimes over one and a half cubic yards of concrete were secured for each barrel of cement used. This large percentage was obtained by the use of large masses of stone completely embedded in the primary concrete.

Rubble concrete was extensively used for the construction of a dam and accompanying masonry work for the Jersey City Water Supply Co., at Boonton, N. J. This concrete was placed under contract at a cost of $1.98 per cubic yard. The methods used were similar to those described above. The entire work, as far as possible, was done by machinery, only a few laborers being necessary to operate the various machines, derricks, etc., and to dump the concrete and joggle the stones in place. The concrete used consisted of one part Portland cement, 2¾ parts sand and 6¾ broken stone, the latter being crusher run up to 3 ins. in size. This masonry contained about 55 per cent. rubble.

Tools for Mixing, Conveying and Ramming.—The tools necessary for mixing and depositing concrete will vary with the

Fig. 37.—Iron Wheelbarrow for Handling Concrete.

number of men working and the kind of work to be done. Square-pointed shovels are usually employed for mixing and handling the materials. Size No. 3 is usually employed. Iron wheelbarrows (Fig. 37) and push carts are used for moving the materials.

Figure 38 shows a Ransome push cart for transporting concrete. This cart is built entirely of steel, and, as it has large

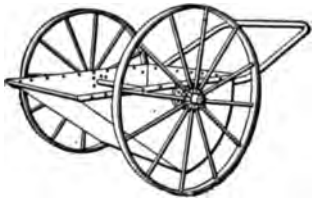


Fig. 38.—Ransome Concrete Cart.

wheels, is very easy to move about, enabling one man to move several times as much material as he could handle with a wheelbarrow. This form of cart is easy to dump, and concrete is not readily spilled over the sides of the bowl. This saves both time and material. This cart may in many cases be used with profit to replace the old-fashioned wheelbarrow. It is stated by the makers that the cost of moving concrete with a Ransome push cart is 1¼ cents per cubic yard per 100 feet of average haul.

A bottomless measuring box, as described on page 66 (Fig. 20) will be necessary if the materials are properly measured. Hoes are sometimes used for mixing and leveling off in depositing. In no case should rakes be allowed on the work, as by their use the larger stones will be loosened and separated from

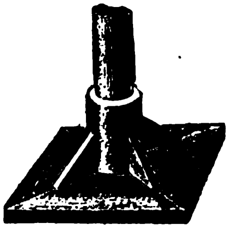


Fig. 39. Iron Rammer for Dry Concrete.

the mass of concrete. Rammers are used for compacting the materials. When dry mixtures are used the style of rammer shown in Figure 39 is employed; while for wet mixtures a rammer of the form shown in Figure 40 may be used to cut and compact the materials.

Fig. 40. Wooden Rammer for Wet Concrete.

For thin walls a tool having a long flat steel blade mounted on a handle like that shown in Figure 41 will be found of use to cut and compact concrete. When placed in thin sections a wedge-shaped rammer will at times be of service. Various shaped ram-

Fig. 41.—Ransome Rammer for Thin Walls.

mers of small section are used for compacting concrete in contracted spaces.

Laying Concrete During Freezing Weather.—As a rule it is undesirable to lay concrete in freezing weather. Certain considerations, however, at times make it not only necessary but imperative to carry on construction work during cold weather. Under these conditions, if proper precautions are taken, the work may be carried on without seriously injuring the concrete.

The effect of freezing upon cement and cement mixtures will be discussed in Chapter IX. In this place we will only consider the precautions which it is necessary to take to carry on this class of construction work in freezing weather.

The precautions which will assist in preventing the injury to concrete during freezing weather are as follows:

1st. Preparation of the concrete materials so as to delay the action of frost.

2d. The protection of the newly-laid concrete, either by covering it with some non-conductive material which will retain the heat, as straw, sand, sawdust, burlap or manure.

3d. By enclosing or housing the work under construction.

4th. The artificial heating of the enclosing space.

The particular method to be used will depend largely upon the character of the work.

When laying mortar in freezing weather as little water as possible should be used in mixing, as the less water used the quicker will be the setting. Soda is sometimes used to hasten the setting. As a rule, however, it is not desirable to add such an ingredient, as there is danger of injuring the strength of the concrete. Hot water is sometimes used in mixing to prevent freezing. Again the materials are heated, and the mix is tempered with hot water. If the concrete be deposited in mass it will retain the heat for seveal hours, giving the cement time to take its initial and final set. In each period there is little danger of injury from freezing.

Salt has been used in varying amounts depending upon the degree of cold to delay the action of freezing.

Heating the Materials.—The method to be adopted for heating the materials will depend largely upon the character of the work and the arrangement of the mixing plant. If the mixing is done by hand, sand and broken stone may be heated by means of an ordinary sand heater, such as is used for heating gravel used in street paving work. Figure 42 shows an apparatus which has been used on the New York Central Railroad for heating concrete materials. This apparatus contains a tank for

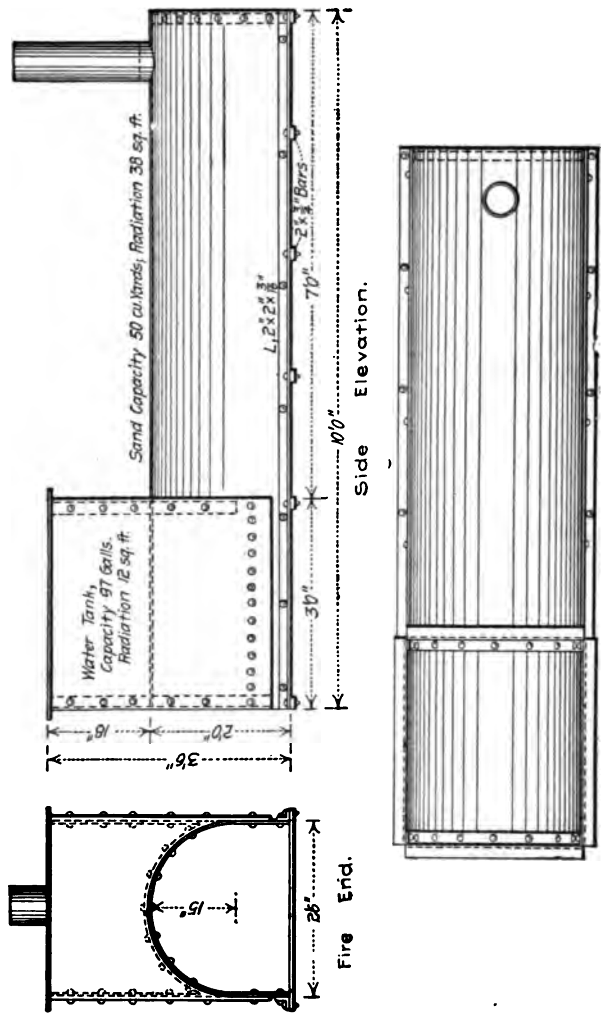


Fig. 42.—Apparatus for Heating Sand, Stone and Water.

heating the water as well as providing heating surface for the stone and gravel. These heaters weigh about 1,200 pounds, and are easily transported from one job to another. It is stated that they can be built at a cost not exceeding about fifty dollars.

For heating concrete materials in cold weather Mr. Wm. H. Ward used the following method: A large watertight tank, open at the top, was constructed of such dimensions as would allow three ordinary dirt boxes to be lowered into it at the same time. This tank was filled with water, and a jet of steam kept the water hot in the coldest weather. The broken stone was loaded into boxes and lowered into the tank of hot water. A few minutes' immersion was sufficient to heat the stone to the desired tempera-

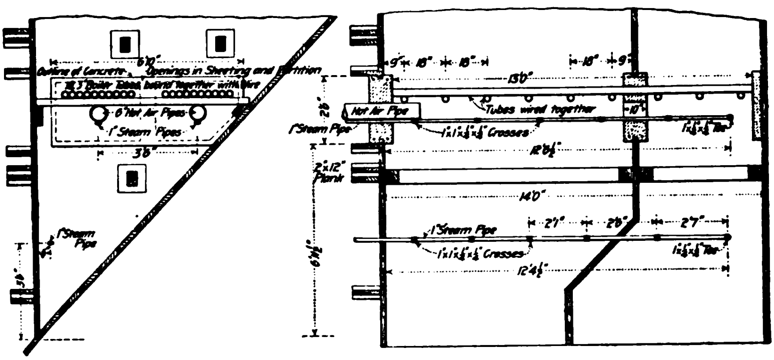


Fig. 43.—Arrangement for Heating Sand and Stone in Bins.

ture, when it was hoisted out and dumped at the mixing machine. The stone was found to retain heat until setting had taken place.

Extremely crude methods may be used successfully where the importance of the work does not warrant any considerable outlay for apparatus.

In the construction of the Foster-Armstrong Piano Company's shops, at Rochester, N. Y., storage bins were provided with apparatus to heat the sand and gravel. Platforms or gratings of tubes were set close together and supported, as shown in Fig. 43. Beneath these cavities V-shaped openings were formed in the sand and gravel. Pipes project through one end of the bin through these cavities from a hot-air furnace and a steam boiler. The hot-air pipes merely pass through the wall, but the steam pipes continue nearly to the opposite side of the bin and are provided with open crosses at intervals along their length. Where the pipes penetrate the wall and partition concrete slabs are inserted. In addition to these there is a small pipe for steam located below and near the bottom of the bin. The hot air pipes connect with a small furnace, and the air was forced through them by a No. 6 Sturtevant blower. The heating power furnished by this apparatus proved sufficient to keep the gravel and stone from freezing, although the top of the bin was open to the weather.

Where only moderate quantities of concrete materials are to be used a bottomless box containing a coil of steampipe may be used for heating the stone and gravel, the heated materials being drawn off from the bottom of the box to the mixing board or machine.

Use of Salt.—Because of its cheapness and the ease with which it may be obtained, salt has been extensively used to lower the freezing point of water. Other materials, such as glycerine, alcohol and sugar, have been experimentally employed, but appear to have a tendency to lower the strength of the mortar.

A common rule for the use of salt is to dissolve one pound of salt in eighteen gallons of water when the temperature is at 32 degrees Fahrenheit, and add one ounce for each degree of lower temperature. Professor Tetmajer's rule, reduced to Fahrenheit units, requires 1 per cent. by weight of salt to the weight of the water for each degree of Fahrenheit below freezing. In the construction of the New York Subway 9 per cent. of salt to the weight of water was used. On the Wachusett dam, during the winter of 1902, four pounds of salt were used to each barrel of cement. For 1 : 3 mortar this corresponded to about 2 per cent. of the weight of the water.

Experiments show that ordinary "quaking" concrete, in proportions of 1 : 2½ : 5, requires about 120 pounds of water per barrel of Portland cement, and 10 per cent. of salt when used in such mixture is equivalent to 12 pounds per barrel of Portland cement. Ordinary 1 : 2½ mortar requires about 120 pounds of water per barrel of Portland cement. This would be equivalent to 12 pounds of salt per barrel of cement. The effect of salt seems to be to increase the time of setting, although if not used in too large quantities no material decrease in strength of the mortar or concrete results.

Protecting Surface With Coverings.—After the concrete has been deposited the heat may be retained for several hours and until final set has taken place by covering the surface with sand, straw, burlap, sawdust or manure. A covering of sufficient thickness should be used. Sawdust when available will probably supply the best protection with little danger of injuring the concrete. Manure, perhaps, will retain heat best and keep the surfaces warmer than the other materials, as the decaying materials, on account of the chemical change, give off considerable heat. Ammonia gases are generated and given off by the action of decomposition and may injure the concrete.

Manure should be used with care for protecting concrete

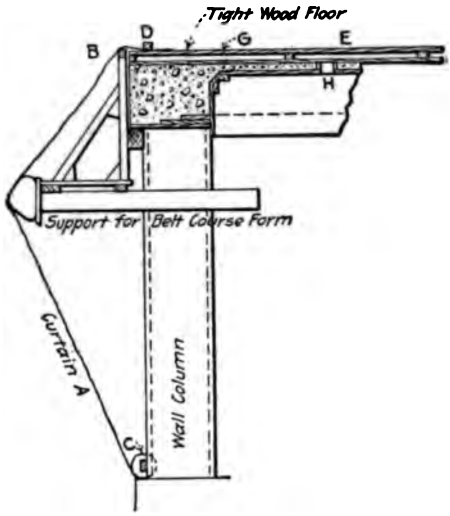


Fig. 44.—Sketch Showing Methods of Enclosing Building with Canvas Curtains.

in freezing weather. When possible the concrete should first be covered with boards and the manure placed upon them. This will keep the manure from soiling the concrete and prevent any action of gases generated by the decaying materials. When there is liability of wet weather, followed by alternate thawing and freezing weather, manure should not be used. It is stated by Mr. Leon D. Conklin, City Engineer, Elmira, N. Y., that a concrete walk covered with manure for about a month crumbled and became badly broken up a short time after the manure was removed. It is probable, however, that only green concrete is injured by manure, as it has been used widely for stable floors, manure bins, etc., without any evidence of failure.

The other materials may be used freely without any attendant dangers.

Housing.—A suitable housing may be used at times for enclosing the concrete work. At Beverley, Mass., the three-story factory building described later, was enclosed in a house of canvas on a light wooden frame, so that the concrete was mixed and laid under cover, while the temperature was maintained at the freezing point by means of stoves.

In the construction of a dam at Chaudriere Falls, Province of Quebec, when the temperature was as much as 20 degrees be-

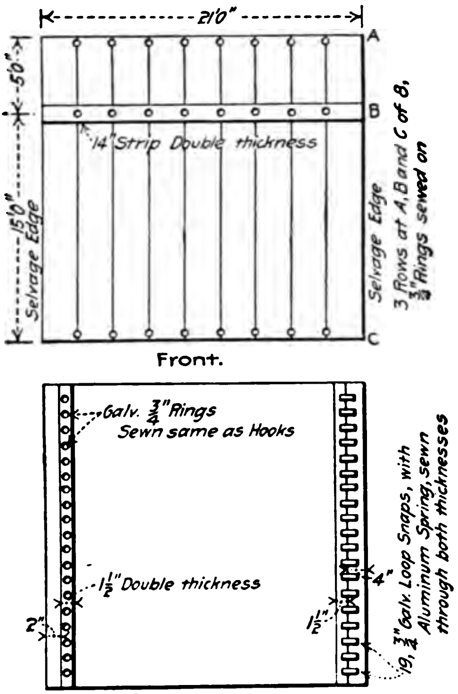


Fig. 45.—Canvas Curtain for Enclosing Buildings.

low zero, a house about one hundred feet long by twenty-four feet wide, was built over a portion of the dam, and heated by iron stoves burning coke. The concrete was mixed and laid in this house. When one portion of the dam was completed, the house was taken down and erected in another place and the work continued.

In the construction of the shops for the Foster-Armstrong Piano Company, at Rochester, N. Y., a special form of housing was used. As the building was constructed a temporary structure of timber and canvas was erected to enclose the exterior walls. The open sides were composed of canvas curtains and the floor covered with timber shutters. The curtain (A), Figs 44 and 45, is held by tying-rings to a continuous string-piece (B), the upper portion, or flap D, being held down by a metal rod or other heavy object, so as to lap over the floor cover (E). At the bottom the curtain is attached the stringpiece (C).

Figure 44 shows how the curtain adjusts itself to irregular

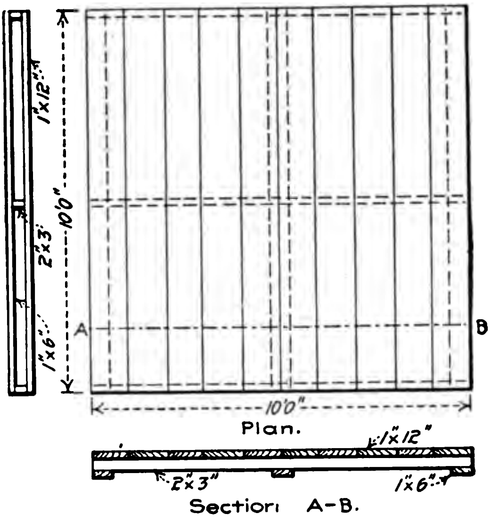


Fig. 46.—Wooden Floor Panels.

projections, such as the support for the belt course forms. To prevent the canvas from tearing on the timbers, these are cushioned by rolls of bagging or other suitable materials. The construction of the wooden floor panels is shown by Fig. 46. They are so designed that a hollow space is left between them and the floor. To provide for the circulation of air in this space, holes are formed through the concrete floor slab, as shown at H (Fig. 44). The drawing (Fig. 46) shows a 10 × 10 ft, panel, made of 12 × 1-in. boards, nailed to the edges of 2 × 3 in. battens; to the opposite edges of the battens were attached 6 × 1 in. bars to stiffen the rods and give a good bearing on the green concrete.

Heating the Enclosed Space.—The space enclosed by the housing was heated by means of coke fires in braziers and by a system of steam pipes from a central boiler. The open fires were scattered throughout the floor area and were simply fires of coke in home-made braziers of reinforcing metal. Each brazier held about five cubic feet of coke, and ten braziers were used for the enclosed floor, 50 × 200 ft. × 13 ft. high. The ten braziers and the steam piping kept the temperature at about 80 degrees Fahrenheit in the enclosed space below the floors, and at about 40 degrees in the space between the top of the floor and the outer covering. These temperatures were maintained when the temperature outside ranged from zero to 10 degrees above.

In the construction, the columns were concreted first, then the belt courses and the floor systems. As soon as the belt courses were completed, the canvas wall curtains were hung and likewise the floors were covered with wooden panels as fast as the concreting was finished. The concrete was deposited regardless of the temperature, it being the theory that when once laid, and whether freezing or not, it would be kept above the freezing temperature by the artificial heating arrangements until it was thoroughly set. The theory worked out perfectly and no damage resulted to the work because of the frost.

In the construction of a number of one-story buildings for the Bush Terminal Co., at South Brooklyn, N. Y., the roof slabs were moulded in sections and allowed to set in a building heated by means of open coke fires, the coke being contained in a metal stove.

Any means of artificial heating, such as stoves, hot air furnaces or steam pipes, may be successfully used in a manner similar to that outlined above.

Depositing Under Water.—In heavy construction concrete is often deposited under water. Reinforced concrete cannot, however, be constructed under water; if this material is to be used for submerged structures it must either be constructed on shore and sunk to place or else the space on the bed to be occupied by the structure must be laid dry by cofferdams, and the reinforced concrete construction be carried on as in the open air, and left until thoroughly hardened before the water is let in.

In laying concrete under water, some means must be used by which it may be laid without the materials becoming separated as they pass through the water. This may be done in several ways, some of which are as follows: (1) The concrete is lowered in large buckets, which have a closed top and a hinged bottom that opens when the bucket reaches the bottom. (2) The concrete may be passed through tubes reaching the bottom, in which case the concrete should completely fill the tube and flow continuously until the depositing is completed; stone grouted in place has also been used for foundation work. (3) Concrete has also been successfully deposited in cloth sacks or paper bags. When cloth sacks are used an open woven cloth, like that used for gunny sacks, should be used. The sacks should be about two-thirds or three-fourths full of concrete, and, when practicable, placed in courses, header and stretcher system, ramming each course as laid. The bagging is close enough to keep the cement from washing out, and at the same time open enough to allow the whole mass to be united into one compact mass. This method has been successfully used for bridge pier foundations. When paper bags are used they are filled with a fairly dry mixture and lowered into place, sometimes by means of a chute. The water soon softens the paper, the pressure of the concrete breaks the bags, and the concrete becomes united into a solid mass.

W. M. Patton recommends that concrete be allowed to take an initial set before placing it in water, as this will prevent the materials from separating and the cement being washed out. The author knows of no case in which this method has been used to any great extent.

Concrete blocks are at times moulded on land and then placed in position by means of a stationary or floating derrick. This method has been extensively used by government engineers for building breakwaters, sea-wall foundations for light-houses, etc. The blocks are usually moulded in large sizes, weighing several tons, at a convenient yard. When it is desired to place them in position they are conveyed to the site of the work and lowered into place by means of a derrick. A diver is usually employed to see that the blocks are lowered into proper position.

Grouting Loose Stone Fill for Foundation.—Concrete for subaqueous foundations may be placed by filling in the foundation area with loose stone or rubble, then sinking at intervals pipes perforated at the bottom and grouting the stone work with a grout made of neat cement paste. This method has been extensively used by English engineers, both in England and India. The grout, on account of its heavy specific gravity, if given sufficient head, replaces the water in the interstices between the stones and firmly cements the stone into one mass of concrete. Neat cement is preferable, as there is a tendency for sand and cement to separate when passed through water. Mr. H. F. White, M. Inst. C. E., states that a 1 : 1 grout was the leanest mortar that could be forced down a 2-in. pipe. In the construction of a breakwater at St. Helier on the coast of Jersey, England, the entire foundation area was filled in with rubble stone and gravel, a diver then sunk the grouting pipes well down into the loose

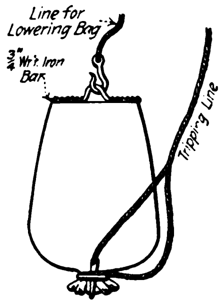


Fig. 47.—Bag for Depositing Concrete Under Water.

stone fill at intervals of 10 to 12 ft. apart. These pipes were 3 ins. in diameter with the bottom end open, and were perforated with ¾-in. holes for 12 ins. above the bottom. The grout was then poured into the pipes and kept flowing until the diver observed the cement coming to the top of the stone. Grouting was then stopped and the pipes placed in new positions. The water varied from 20 to 60 ft. in depth for the St. Helier breakwater foundation.

A Concrete Depositing Bag.—Figure 47 shows a bag for depositing concrete under water. The bag tapers 3 ins. on the side to facilitate discharging. The mouth of the bag is closed by one turn of a line, which is provided with loops through which is a hard wood pin attached to a tripping line. The folds of the bag hold the pin in place while it is being lowered; when in position a sharp pull on the line releases the pin and the concrete is re-

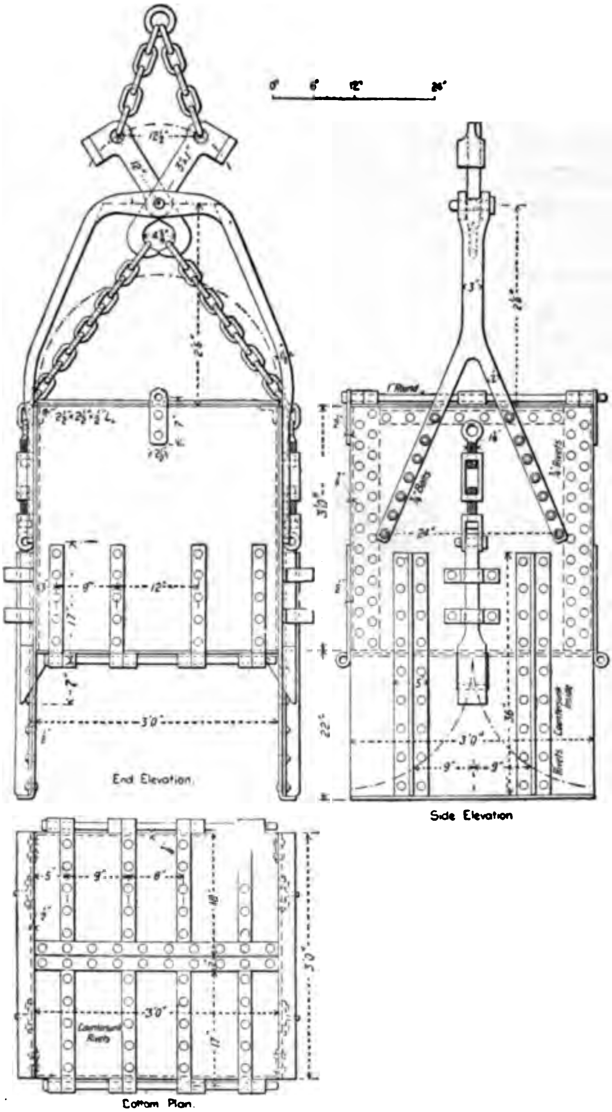


Fig. 48.—Bucket for Depositing Concrete Under Water.

leased. This bag may be made any desired size of canvas or other suitable material and is adapted for use when expensive depositing machinery is undesirable.

Depositing Concrete in Buckets.—Figure 48 shows a concrete bucket used by Prof. W. D. Taylor for depositing concrete under water for foundation of bridge piers for the Coosa River bridge, Louisville and Nashville Railroad. Concrete was deposited as deep as 26 feet. When the water was pumped out of the coffer dams the concrete was found to be very hard, and required very little leveling up over an area of 15 × 38 ft. The bucket holds 1 cu. yd. of concrete and is handled with a derrick. The bucket is

Fig. 49.—Cyclopean Bottom-Dumping Bucket.

so designed that when its sides rest upon the bottom, the "scissors" unhook, releasing the dogs that hold the swinging bottom doors, allowing them to drop. It was found necessary to make the flanges on the bucket wider than shown, to keep them from cutting into the fresh concrete.

In the construction of the foundation for a masonry dock at New Rochelle, N. Y., concrete was deposited at a depth of from 14 to 19 ft. below mean low tide. The concrete was lowered into place by means of a ½ cu. yd. bottom dump Cyclopean bucket. The bucket was lowered into the water by a derrick operated by a Lidgerwood hoisting engine on a scow, and dumped when near the bottom by means of a line operated by a man on the scow. Very little cement was washed out of the concrete when the bucket was submerged. To provide for any possible waste, 25 per cent. excess of cement was used in mixing the concrete.

Figure 49 shows the style of Cyclopean bucket most often used, while a style especially adapted for use in sub-aqueous work is shown open and closed in Figs. 50 and 51.

In the construction of the foundation for the South Pier at Superior Entry, Duluth Harbor, Minn., a steel bucket, so designed that after it had been set upon the bottom, it was tripped by a special designed latch from which a rope led to the derrick man, was used. The bucket was covered with canvas covers or curtains quilted with sheet lead and fastened to opposite sides of

Fig. 50.—Cyclopean Subaqueous Bucket, Closed.

Fig. 51.—Cyclopean Subaqueous Bucket, Open.

the buckets. When in position the curtains overlap at the middle of the bucket, completely covering the exposed concrete. It is stated by U. S. Engineers in charge of this work that these covers proved entirely satisfactory. Covers of this kind could with little difficulty be attached to the Cyclopean and similar buckets, thereby further protecting concrete when placed under water.

The O'Rourke Bucket.—A similar bucket designed and patented by John F. O'Rourke, M. Am. Soc. C. E., was used in the construction of the foundations for the City Island Bridge in New York City. Figure 52 shows the bucket closed to carry its load of concrete. It is rectangular in form, with flap doors at the top, while the bottom is left open. The timber frame at the bottom gives the bucket a wide bearing when it rests upon the soft deposited concrete, and prevents it from sinking into and cutting up the concrete. For holding the concrete in the bucket there are two interior flap doors, which, when closed, form a V-shaped interior hopper bottom, and when open swing back against the sides of the shell, and leave the bucket open its full bottom dimensions. The doors are held closed by chains attached to the bail, which is held by the key or pin shown just above the chain connections to the bail. When filled, the bucket is swung clear of the ground, and the pin or key is pulled out,

Fig. 52.—O'Rourke Concrete Bucket, Closed.

leaving the pull of the load on the bail to hold the doors closed. In this condition the bucket is swung over the spot where it is desired to deposit its load, and then lowered until it reaches the bottom. As the bucket comes to rest, the load of concrete on the doors pulls on the chains, and this pull, added to the weight of the bail, which is purposely made very heavy, causes the bail to slide down into the position shown by Fig. 53, and also causes the door to swing open as shown. As the doors swing open they tend to force the water out of the shell, while the shutter attached to the door closes the slots, in which the pins at the ends of the doors slide back and forth. As soon as the doors have swung clear open the latches on the bail catch as shown in Fig. 53, and hold the bail from sliding up until the latches are released by hand.

This bucket thoroughly protects the concrete until the moment of dumping, discharges automatically with the largest possible opening for the size of the bucket, and is extremely simple in construction. The discharge of the load is effected simply by raising the bucket free from it.

Depositing by Chutes.—A tube or chute, sometimes called a trémie, is used at times for depositing concrete under water. It

Fig. 53.—O'Rourke Concrete Bucket, Open.

consists of a tube open and usually flared at the top, to receive the concrete. The tube is built in detachable sections, so that its length may be adjusted to the depth of water. The tube is suspended from a crane upon a track so that it can be moved about as the work progresses. The upper end is kept above water, while the lower end sets upon the bottom. The trémie is first filled by placing its lower end in a box with a movable bottom filling the tube, lowering it to the bottom, and then detaching the bottom of the box. If it is not convenient to first fill the tube, it is lowered to the bottom and filled by dropping the concrete in the tube. When this method is used the first charge of concrete is lost.

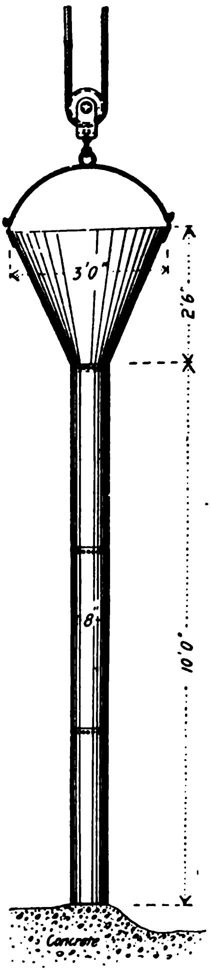


Fig. 54.—Tremie or Tube for Depositing Concrete Under Water.

Figure 54 shows a tube of this character used by Mr. Wm. H. Ward in the construction of the Harvard bridge foundations, for the foundations of the Brooklyn Heights Power House, and for a number of other structures the depths of water varying up to 18 ft. This system was also used in constructing the foundations of the Boucicault Bridge over the Saône River in France. A chute of this kind was used in the constuction of the foundations for the piers of the Charlestown Bridge, Boston, Mass. At this place the piers were enclosed with cofferdams and the concrete deposited on piles driven to such a depth as to secure a suitable foundation.

The chute was a tube 14 ins. diameter at the bottom, and 11 ins. at the neck, with a hopper at the top to receive the concrete. This tube was made in removable sections with outside flanges to adapt it to different depths. It was suspended by a differential hoist from a truck that moved laterally on a frame, which traveled the length of the pier. (See Fig. 55.)

In operation the foot of the chute rested on the bottom and concrete was dumped into the hopper. The chute was then raised and the concrete allowed to run out in a conical heap, the loss being made good by dumping in more concrete at the top. By moving the truck on the traveler, a ridge of concrete was deposited across the pier, the chute always being kept full or nearly so. When a ridge was finished the traveler was moved and another ridge built, this operation being continued until the whole surface was covered. The thickness deposited depended upon the height to which the foot of the chute was lifted above the bottom. Concrete was laid up to 6 ft. in thickness, but it was found that the best results were obtained when layers about 2½ ft. in thickness were deposited. If the chute was raised too high or too quickly, a charge was lost.

Fig. 55.—Mounting for Tremie, Charlestown Bridge Work.

The chute seemed to work best when the concrete was mixed not quite moist enough to be plastic. If mixed too wet, the charge was liable to be lost; if too dry, there was a tendency to choke the chute.

CHAPTER VII.

COST OF CONCRETE.

The cost of concrete varies widely, depending upon the character of the construction, local conditions, and the cost of materials and labor. For ordinary work, when concrete is laid in large masses, the cost per cubic yard will vary from $3 to $7. For pavement foundations in Brooklyn, N. Y., when competition was sharp, bids as low as $3 a cu. yd. for a 1 : 3 : 6 mixture, thickness about 6 ins., secured the work. Under usual conditions in New York and Brooklyn for this class of work, the bids average from $4.50 to $6.50 per cu. yd. For footings for wall foundations in building construction in New York City, a common price is 25 cts. a cu. ft., or $6.75 per cu. yd., for a 1 : 3 : 5 mixture, where the yardage is small. When concrete is laid in thin sections, as in sewers, small arch bridges, thin walls, etc., costs will range from $7.00 to $15.00 per cu. yd., including cost of centering.

For reinforced concrete work, as in buildings, when thin slabs, beams and columns are used, the cost, including forms, finish, etc., will vary from $10.00 to $22.00 per cu. yd. The cost will be found to be nearer the higher figure when first-class work is insisted upon.

The items to be considered in figuring the cost of concrete are as follows:

(1). Cost of cement, sand and broken stone or gravel delivered at the work.

(2). Cost of loading the barrows, buckets, carts or cars used to convey the materials to the mixing board or machine.

(3). Cost of transporting and dumping materials.

(4). Cost of mixing by turning with shovels or by machine.

(5). Cost of loading concrete into barrows, buckets, carts or cars.

(6). Cost of transporting the concrete to place.

(7). Cost of dumping and spreading.

(8). Cost of ramming.

(9). Cost of forms.
(10). Cost of runways, cement house, bins, platforms, etc.
(11). Cost of finishing the surface.
(12). Cost of superintendence and general expense.
(13). Interest on capital invested and depreciation of plant.

The cost of cement will vary with market prices, freight rates and cost of transportation to the work.

The first two items must be determined for each piece of work. The cost of transportation will be 15 cents a ton-mile, assuming team wages at $3.75 a day, and length of haul 10 miles, the wagon going one way empty. This gives a cost of 3 cents a barrel of 400 lbs. per mile of haul.

Cost of Sand.—The cost of sand, under exceptional conditions, may be as low as 20 or 30 cents per cu. yd. delivered at the mixer, but under usual conditions will be found to range from 50 cents to $1.00 per cu. yd. When sand is very difficult to obtain, and must either be brought from a long distance or made by crushing rock, the cost may range from $1 to $3 per cu. yd.

The cost of hauling sand will depend somewhat upon its unit weight. The weight will depend upon the character of rock from which it is made and upon its physical condition. Sabin* gives the following data in regard to the weight of sand: Natural sand, as it ordinarily occurs, will weigh about as follows, according to its condition:

	Lbs. per cu. ft.
Moist and loose	70 to 90
Moist and shaken	75 to 100
Dry and loose	75 to 105
Dry and shaken	90 to 125
When settled in water weight of wet sand, voids full	100 to 140

If the rock from which the sand is made weighs, say, 160 lbs. per cu. ft. solid (sp. gr. 2.56) the sand will weigh per cubic foot 120, 100 and 80 lbs. for voids of 25, 37.5 and 50 per cent. respectively.

Assuming the weight at 100 lbs. per cubic foot, one cubic yard will weigh 2,700 lbs., and the cost for transportation in wagons at 15 cents per ton-mile as given for cement, will make the cost for transportation 20 cents per cu. yd. per mile.

Cost of Gravel and Stone.—The cost of gravel or broken stone

*Cement and Concrete, New York, 1905.

will depend upon the locality. Under exceptional circumstances when the gravel pit is near the work, the gravel may be delivered at the mixing platform for from 25 to 40 cts. per cubic yard, but usually it must be brought from a distance, and will cost from 60 cts. to $1.00 per cu. yd., or more.

The cost of broken stone will depend upon the cost of quarrying, crushing and delivering at the work. The cost of quarrying and crushing will, under average conditions, vary from 50 cts. to $1.00 per cu. yd. See also page 47. Therefore stone delivered at the work will cost from, say $1.00 under good conditions to $2 or $3 per cu. yd. under less favorable conditions.

Total Cost of Materials.—Let us assume that a 1 : 2 : 4 mixture is to be used, and that cement costs $1.60 per bbl. delivered, sand $1.00, gravel $1.00 and broken stone $1.50 per cu. yd. delivered.

Referring to Table IV., page 54, we find that the quantities of materials required for 1 cu. yd. of gravel concrete are 1.34 bbls. cement, 0.41 cu. yds. sand, and 0.81 cu. yds. of gravel. The cost estimate will be:

Cement, 1.34 at $1.60	$2.14
Sand, 0.41 at $1	.41
Gravel, 0.81 at $1	.81
Total cost for materials per cu. yd.	$3.36

For broken stone concrete, referring to Table IV., page 54, the quantities of materials when stone 1-in. and under are 1.46 bbls. cement, 0.44 cu. yds. sand, and 0.89 cu. yds. stone. The cost estimate will be:

Cement, 1.46 at $1.60	$2.34
Sand, 0.44 at $1	.44
Stone, 0.89 at $1.50	1.33
Total cost for materials per cu. yd.	$4.11

The cost of the materials for a cubic yard of concrete will, of course, vary for other proportions as well as for other costs for raw materials.

The cost of loading barrows, buckets or carts is as follows: One man should shovel 15 cu. yds. of sand, gravel or broken stone into a barrow in 10 hours. With good supervision, and a willing worker, the amount may be increased to 20 cu. yds., but for poor supervision it may be less than 15 cu. yds.

Assuming wages at $1.50 for a 10-hour day, and that 15 cu.

yds. are handled by one man, this gives us a cost of 10 cents per cu. yd. For the 1:2:4 broken stone mixture given above the cost of loading will be:

Stone, 0.89 cu. yd.	8.9 cts.
Sand, 0.44 cu. yd.	4.4 "
Cement, say	2.0 "
Total for 1 cu. yd. of materials loaded in barrows or carts	15.3 cts.

Cost of Transporting and Dumping Materials.—This cost item will, of course, depend upon the length of haul and whether wheelbarrows, carts, cars or some other means of transportation are used. When possible the stock piles should be so located that the materials will not have to be wheeled up hill. When wheeling on a level with a plank runway, one man can wheel about 3 cu. ft. of sand or stone, while if he must wheel up a steep rise about 2 cu. ft. will be about the maximum load handled. Gillette* states that "a man wheeling a barrow travels at the rate of 200 ft. per minute, going and coming, and loses ¾ minute each trip dumping the load, fixing runway planks, etc. An active man will do 20 or 25 per cent. more work than this, while a lazy man may do 20 per cent. less. With wages at 15 cts. per hour, the cost of wheeling the materials for one cubic yard of concrete may be obtained by the following rule: To a fixed cost of 4 cts. (for lost time) add 1 cent for every 20 feet of distance from stock pile to mixing board, if there is a steep rise in the runway; but, if the runway is level, add 1 cent for every 30 feet of haul."

When the distance to be hauled is more than 50 feet, the materials can be transported at a less cost in one-horse dump carts. A cart should be loaded in 4 minutes and dumped in about 1 minute, making 5 minutes lost time each round trip. It should travel at a speed of not less than 200 feet per minute. A cart should carry about ½ cu. yd. of stone or sand, and, at a cost of 30 cts. per hour for wages of horse, cart and driver, the cost of hauling materials for 1 cu. yd. of concrete is determined by Gillette* from the following rule:

"To a fixed cost of 5 cts. (for lost time at both ends of the haul) add 1 cent for each 100 feet of distance from the stock piles to the mixing board."

*Hand Book of Cost Data, N. Y., 1905.

With carts it will be found that the stock piles may be located some distance from the work without materially increasing the cost of haul.

Cost of Hand Mixing.—The cost of mixing concrete by hand will depend upon the number of times the materials are turned over with shovels. A good worker should turn over mortar or concrete at the rate of 3 cu. yds. per hour, lifting each shovelful and casting it into a pile. This is at the rate of 5 cts. per cu. yd. per turn, wages being 15 cents per hour. The cement and sand forming the mortar will measure about 0.4 cu. yd. per yard of concrete. If 6 turns be given to these before adding the stone, we have for the cost of turning the mortar $0.4 \times 6 \times 5$ cts. = 12 cents. If the stone and mortar are turned 3 times, we have $1 \times 3 \times 5$ cts. = 15 cts., or the total cost of turning is 12 cts. + 15 cts. = 27 cts. per cubic yard. If more or less turns be given to the materials the cost will, of course, vary accordingly.

The amount of work done by each mixing gang will depend upon the supervision, a good foreman at times getting 50 per cent. more work out of the men than a poor one.

Gustave R. Tuska* states that the cost of mixing alone of the concrete for the Lonesome Valley Viaduct, colored labor being employed at $1.00 per day of 11 hours, was 8 cts per cu. yd. This is exceptionally low, for assuming wages at 15 cts. an hour as above, the cost would have been 13.2 cts., less than half of the cost given above; but the mixing may not have been as thorough.

The Cost of Machine Mixing.—This will depend upon the kind and size of mixer used. With the old style of cubical mixer, which has for a long time been used on government work, the cost of mixing as given in Government Reports is as follows: On the Buffalo breakwater, mixing, including engine men and derrick men, was 12.9 cts. per cu. yd. For another piece of work, when the mixing was done with a Chicago improved cube mixer, the cost of mixing alone was 2.73 cts. per cu. yd. Ransome states that he has mixed concrete at a cost as low as 2 cts. per cu. yd.

Hence it will be seen that considerable economy will result if machine mixing be used. Assuming a cost for machine mixing at 10 cts., and for hand mixing 27 cts., as given above, a saving of 17 cts. per cu. yd. will result if a machine be used. If the

*Transactions Am. Soc. C. E. Vol. XXXIV., p. 247.

cost be as low as 7 cts., a saving of 20 cts. will result. Assuming that a mixing machine with a capacity of 100 cu. yds. per day will cost $800, the machine will pay for itself if 4,000 cu. yds. of concrete are mixed, or in 40 working days. Any concrete mixed in excess of 4,000 cu. yds. will yield 20 cts. per cu. yd. clear profit to the contractor, besides having his machine paid for.

Cost of Loading and Transporting Concrete.—The cost of loading concrete after it is mixed is less than the cost of loading the raw material before mixing, as the volume of the concrete is less than the volume of the unmixed ingredients. This cost should not exceed 12 cents per cu. yd. When the mixing is done by machinery the cost should be practically nothing, as the concrete is dumped directly into the barrows or carts.

The cost of transportation will be about the same as was given for transporting the raw material.

Where hoisting machinery is used for elevating the concrete from the machine, as in a building, the concrete is dumped directly into the hoisting bucket or conveyor, which is usually self-dumping. The expense of hoisting will only be that of the power used, which should not exceed from 1 to 5 cents per cu. yd., depending upon conditions.

Cost of Dumping and Spreading.—The cost of dumping is usually included in the cost of transportation. The cost of spreading will depend upon the consistency of the mixture, a dry mixture requiring considerable work, while little attention is needed for a wet mixture. The cost when a wheelbarrow is used for transportation will probably not exceed 4 or 5 cents per cu. yd., while for carts, owing to the larger amount dumped in one place, the cost is probably double this, or somewhere near 10 cents per cu. yd. These figures are for street paving, foundations and similar work, where the materials are placed in layers 6 or 8 ins. in thickness.

Cost of Ramming.—The cost of ramming depends upon the dryness of the mixture and number of cubic yards delivered to the rammers. A dry concrete needs a great deal of ramming to compact it so that water will flush to the surface, while a wet concrete can not be rammed at all, and need only be spaded a little to release imprisoned air. The amount of concrete delivered affects the cost of ramming, as if considerable concrete is delivered the men must keep quite busy to ram it properly in place; and, when the amount is smaller, there is a tendency for the men to soldier. The ramming of a moderately dry concrete should not cost more than 15 or 20 cts. per cu. yd., although at times it may be as high as 40 cts.

Cost of Forms.—Forms may vary from the simplest to very elaborate construction in wood, requiring skilled and costly carpenters' work. No general cost figures can be given. The most satisfactory way of arriving at their cost is to estimate the amount of lumber required. Then secure a carpenter's estimate of the cost of framing them as desired. This cost may be as low as $4 or $5 per 1,000 feet board measure, and will at times greatly exceed this when the framing is intricate.

The usual way of estimating the cost of forms is to make an allowance per cubic yard of concrete to be laid. Some cost data will be given in connection with descriptions of various classes of work in later chapters of this book.

Summary of Cost.—Local conditions will govern the outlay necessary for storage sheds, bins, runways, platforms, etc. Cost of finishing will depend upon the kind of finish used. Some data are given in another chapter in connection with descriptions of methods of finishing.

Cost of superintendence will depend upon the amount of work to be done and the organization used for the work. For average sized jobs this item should not exceed 10 per cent., and for large jobs it may be much less.

Summarizing the above items to determine the cost of concrete for a street paving job, we have for cost of labor:

	Hand Mixing.	Machine Mixing.
For loading cement, sand and stone	$0.15	$0.15
Wheeling 40 ft. in barrows (4 + 2 cts.)	.06	.06
Mixing	.27	.07
Loading concrete into barrows	.12	...
Wheeling 60 ft. (4 + 2 cts.)	.06	.06
Spreading and ramming	.20	.20
	$0.86	$0.54
Superintendence, 10%	.09	.05
Total cost of labor	$0.95	$0.59
Cost of materials, stone concrete,	4.11	4.11
Total cost per cu. yd.	$5.06	$4.70

CHAPTER VIII.

FINISHING CONCRETE SURFACES.

Methods of Finishing Concrete Surfaces.—Special precautions are necessary to secure a good finish to the exposed faces of concrete work. In Europe it is customary to finish the surfaces with a thin coat of mortar, a 1 : 2 or 1 : 3 mixture being floated or troweled on the rough surface of the concrete as soon as the forms have been removed, and while the concrete is still green. In no case should a plaster finish be used, as sooner or later it will shell off, leaving the surface most unsightly.

Where a mortar facing is desired, it is customary to make it from 1 to 1½ in. in thickness, although it is sometimes made as thin as ½ in., or as thick as 3 ins. The mortar facing is usually composed of 1 part Portland cement to 2 or 3 parts of sand. When a special glossy finish is desired, a 1 : 1 mixture is used. However, the best results will be obtained when the ratio of sand to cement in the mortar is the same as that in the concrete. When a granolithic surface is desired, grit or crushed granite containing no particles greater than ⅜ to 1 in. is used, in place of sand, and 1½ to 2½ parts stone to 1 of cement are used.

The facing may be applied in several ways. A layer of mortar an inch or so in thickness may be troweled against the face of the form, and the concrete immediately deposited against it, holding it in place. By throwing the concrete against the face of the moulds with considerable force the larger stones will rebound and the mortar remain against the mould. Another, and more common method, is to force a spade down the side of the moulds, pushing back the stones and allowing the mortar to flow in against the mould face. Sometimes a broad, flat shovel is used and mortar poured down between it and the mould.

Another method by which a definite thickness of mortar facing is secured is to place a mould like that shown in Fig. 56 against the form and fill the space between it and the mould with mortar. This mould consists of a sheet-iron plate 6 or 8 ins. wide and 5 or 6 ft. long, having riveted across it on the side which faces the mould small angles the size of the thickness of facing desired, usually $1\frac{1}{2}$ x $1\frac{1}{2}$ ins., and spaced close enough together to support the plate. The upper edge of the plate is flared, as shown, to assist in placing the mortar, and it has two handles to facilitate its removal. This form is placed with the projecting legs of the angles against the face of the mould and forms with the mould an open space about $1\frac{1}{2}$ in. wide. The space is filled with the facing mortar, which is lightly tamped; the concrete is then filled in behind the mould; the iron mould is then withdrawn, and the whole mass thoroughly tamped. The mortar is mixed in small batches and deposited with shovels. Care should be taken not to mix the mortar too wet, or the larger stones in the concrete will be forced through the mortar face against the form when the

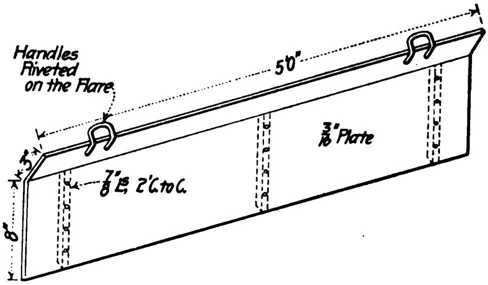


Fig. 56.—Mould for Applying Mortar Facing.

mass is tamped. To secure a satisfactory bond, the mortar and concrete should be placed at the same time.

Treating a Pitted or Mottled Surface.—Whether or not a special mortar facing is used, the surface of the concrete will often contain pittings, bubble-holes, rough spots and grain marks from the wood of the mould. A mottled appearance due to variations in the color of the sand, etc., may also occur, and some special treatment is necessary to secure a uniform and pleasing finish.

Immediately after the forms are removed, the surface should be cleaned of any grease or oil from the forms. Any small depressions or holes may then be filled with mortar well rubbed in; projections or ridges due to holes or joints in the forms are rubbed down, and the entire face may then be washed with a grout of 1 part Portland cement and 2 parts fine sand. This should be applied with a brush.

A pleasing finish may be obtained by using plaster of Paris in place of the sand.

A mixture of equal parts of Portland cement and plaster of Paris gives a very light grey shade, and 3 parts of cement to 1 part of plaster of Paris gives a darker shade.

A Rubbed Finish.—This may be obtained by removing the forms before the concrete has set very hard, and rubbing the surface with a circular motion with a white fire-brick or a wooden float. The time the forms are allowed to stand before removing will vary from 12 to 48 hours, depending upon the cement, amount of water used in mixing and the state of the weather. Before rubbing, any voids in the surface should be filled with mortar well rubbed in.

If the concrete is quite green, an effect similar to rubbing is obtained by brushing the surface with brooms or stiff brushes. When the concrete is not green enough to dress easily, water may be used when dressing with the brush, and water and sand with the wooden float.

A very satisfactory surface which has the appearance of cut stone may be obtained where a surface mortar of cement and crushed stone is employed. The color and texture of the crushed stone affects the appearance of the surface, special surfaces being obtained by the use of red or grey granite, sandstone, etc. It is customary to use for the facing a coat of 1 : 2 or 1 : 3 mortar. After the forms are removed the mortar covering the face of the particles of crushed rock is removed by brushing or washing the surface with a weak solution of acid. The surface is then washed with clean water and finally with an alkali solution to neutralize the action of the acid. This treatment leaves the granular parts of the stone partly exposed and gives to the finished work a surface which is difficult to distinguish from natural stone. The size of crushed stone used in the facing will depend upon the character of the finish desired. Usually it should pass through a sieve of 10 to 30 meshes per inch.

Pebble Dash Facing.—A unique surface finish which may sometimes be used with good effect is secured by using rounded pebbles in place of the usual stone for the surface layer of the concrete. After the forms are removed, and while the concrete is still soft, the cement and sand on the exposed face are removed until about half of the surface of the pebbles is uncovered. This finish was used in the construction of a reinforced concrete bridge in the National Park at Washington, D. C. It was found that the pebbles were brushed loose 12 hours after the concrete was laid, and at 36 hours the mortar became so hard that it was removed with difficulty. The brushing was most successfully done when the concrete was about 24 hours old. The mixture employed consisted of 1 part Portland cement, 2 parts sand and 5 parts gravel between 1½ and 2 ins. in the smallest diameter. The cost of brushing was about 60 cts. per sq. yd., or nearly 7 cts. per sq. ft.

Tool-Dressed Surfaces.—When the surface of the concrete has set so hard as to prevent its being treated by any of the methods already described, it may be tool-dressed by any of the methods employed for dressing stone. This is usually done either by hand or by the use of the pneumatic hammer. Hand work consists

Fig. 57.—Ransome Ax for Dressing Concrete Surfaces.

usually of bush hammering or pointing with a chisel. Bush hammering may be done by ordinary labor at a cost of about 1½ cts. per sq. ft., as a man can bush hammer 100 sq. ft. in a 10-hour day. The Ransome concrete ax (Fig. 57) may be used to give a hammer dress finish to the surface of the concrete. It consists of a double-bit ax, having steel blades bolted to a casting in which the handle is inserted. The blades may be removed when dull and are sharpened with a file or an emery wheel. It is stated that a common laborer will average 100 sq. ft. of wall surface in ten hours with a Ransome ax, at a cost of 1½ cts. per sq. ft. From 400 to 500 sq. ft. may be covered in a day with a pneumatic hammer, however, using a special pointed tool.

Grooves are sometimes moulded in the face of the concrete, dividing it into imitation rectangular blocks resembling stone masonry. This is done by nailing triangular strips on the face of the moulds. The same effect is obtained at a greater expense by chisel cutting. A chisel draft is at times cut an inch or two in width about these grooves. The concrete within these pitch lines may be roughly dressed in any manner desired. It is sometimes customary to remove the forms while the concrete is still green and pick over the whole surface rapidly with light picks. This gives an imitation of rough dressed stone. One man can pick over about 100 sq. ft. per day. By choosing the stone for the facing carefully, a very close imitation of natural stone may be secured. Granite crushed to the size of buckshot, or fine gravel, if carefully graded and selected for color, gives a very fine surface which will closely resemble natural stone.

Colors for Concrete Finish.—Coloring matter may be added to the cement to produce imitation stone of various colors. Lampblack is employed to give various shades of grey, according to the amount used. Dry mineral colors, mixed in the proportion of 2 to 10 per cent. of the amount of cement, gives various shades of the colors used. The following colors have been used without any apparent injurious effect: Lampblack (boneblack), Prussian blue, ultramarine blue, yellow ochre, burnt umber and red iron ore. Red lead is injurious, even in so small quantities as 1 per cent., and greater amounts should never be used. Common lampblack and Venetian red should not be used, as they are apt to run and fade. It has been found that ultramarine blue does not affect the strength of the mortar if not used in excessive quantities. Other coloring matter should be used in moderate quantities. Germantown lampblack is also a good material to use on account of the small quantity it is necessary to use to secure good color. The color of a mortar or concrete will vary with the color of the cement, sand and stone used; the color of these ingredients will also affect the final color when coloring matter is used. To produce a colored mortar the coloring matter should be thoroughly mixed with the cement, the sand then added and the whole thoroughly mixed dry, and when stone or gravel is to be used it should be incorporated in the mixture dry, the whole mixed until of a uniform color and then water added gradually, the mixing being continued until the proper consistency is obtained. Table XVIII. gives the usual proportion by weight of different coloring matters to be added to 1 sack of cement and 2 cu. ft. of sand (a 1:2 mixture) to secure different colored mortars.

TABLE XVIII.—COLORING MATTER FOR CEMENT MORTAR.

Weight of coloring matter to 1 sack of cement for a 1 : 2 mixture.

For white stone:

White Portland cement, 1 part;
Pulverized lime, ¼ part;
Pulverized marble, ½ part;
Light colored sand, 1 part.

On account of the inferiority of white Portland cement the above is seldom used.

For black stone:

3 lbs. Excelsior carbon black, or
11 lbs. manganese dioxide.

Grey stone:

1 lb. Excelsior carbon black, or
½ lb. Germantown lamp black (bone black).

Brown stone:

4 to 5 lbs. brown ochre, or
6 lbs. roasted iron oxide, best quality.

Buff stone:

4 lbs. yellow ochre.

Red stone:

5 lbs. violet iron oxide (raw).

Bright red stone:

From 5½ to 7 lbs. English or Pompeiian red.

Yellow stone:

5½ lbs. ochre.

Green stone:

6 lbs. of greenish blue ultramarine blue.

Blue stone:

2 lbs. ultramarine blue.

Dark blue stone:

4 lbs. ultramarine blue.

Purple stone:

5 lbs. Prince's metallic.

Violet stone:

5½ lbs. violet oxide of iron.

In the construction of six emplacements at Fort Wadsworth, New York, the exterior surface was coated with colored mortar mixed according to the following formulas:

For green color:

Cement, 1 bbl.;
Sand, 2 bbls.;
Ultramarine blue, 50 lbs.;
Yellow ochre, 73 lbs.;
Soft soap, 7 lbs.;
Alum, 7 lbs.

For slate color:

Cement, 1 bbl.;
Sand, 2 bbls.;
Lampblack, 50 lbs.;
Ultramarine blue, 35 lbs.;
Soft soap, 7 lbs.;
Alum, 7 lbs.

After completion of the batteries, the color became much lighter with age. It was found that spraying with linseed oil very materially deepened its shade.

Table XIX. is given by Sabin* for various colored mortars.

TABLE XIX.—TABLE SHOWING COLORS GIVEN TO PORTLAND CEMENT MORTARS CONTAINING TWO PARTS YELLOW RIVER SAND TO ONE CEMENT.

Cost of coloring matter per lb. ct.	Dry material used.	Weight of Dry Coloring Matter to 100 pounds of Cement.			
		½ pound.	1 pound.	2 pounds.	4 pounds.
15	Lamp Black	Light Slate	Light Grey	Blue Grey	Dark Blue Slate
50	Prussian Blue	Light Green Slate	Light Blue Slate	Blue Slate	Bright Blue Slate
20	Ultra Marine Blue	....	Light Blue Slate	Blue Slate	Bright Blue Slate
3	Yellow Ochre	Light Green	....	....	Light Buff
10	Burnt Umber	Light Pinkish Slate	Pinkish Slate	Dull Lavender Pink	Chocolate
2½	Venetian Red	Slate, Pink Tinge	Br'g't Pinkish Slate	Light Dull Pink	Dull Pink
2	Chattanooga Iron Ore	Light Pinkish Slate	Dull Pink	Light Terra Cotta	Light Brick Red
2½	Red Iron Ore	Pinkish Slate	Dull Pink	Terra Cotta	Light Brick Red

When a wet mixture is used, the color should appear several shades darker than will be required, as wet mortar looks darker than it really is, owing to the gloss of the water.

As a rule, light shades should be chosen for artificial stone work, as dark colors are contrary to nature, stone in its natural state being of light color and shade. Coloring matter, however, should be used conservatively, as there is more or less liability of the colors fading with time.

It will be found that by varying the amount of water used for mixing ordinary mortars and concretes different shades of concrete can be obtained. Again, by the use of colored sands, different colored concretes will result. Concretes thus obtained are to be desired over artificially colored ones, as the shades approximate more nearly those found in nature.

Painting Concrete Surfaces.—In some cases concrete surfaces may be colored by painting. Ordinary paints are sometimes used, but will not, as a rule, prove satisfactory. When such paints are used it is customary to wash the surface of the wall with dilute sulphuric acid, 1 part of acid to 100 parts of water, before applying the paint.

A grey finish may be obtained by painting with a thin grout

*"Cement and Concrete," Louis Carlton Sabin.

of cement and plaster of Paris. The sheathing should be removed as soon as possible, the surface cleaned from any oil or grease, and the grout applied with a whitewash brush. A mixture of equal parts of Portland cement and plaster of Paris gives a very light grey finish, and 1 part of plaster of Paris to 3 parts of cement gives a trifle darker shade. Similar methods may be used with dry mineral colors. One pound of red iron ore to 10 lbs. cement, mixed dry, and then made into a very thin grout and applied to a clean concrete surface gives a pleasing brick-red color. A rich dark red may be obtained by using 1 lb. of red iron ore to 3 lbs. of cement. The greener the concrete is when any of these preparations are applied the more likely they are to be permanent. In any event, such treatment should be more permanent than ordinary paint.

Masonry Facing has been frequently employed on reinforced concrete bridges, and gives a very satisfactory surface. Any kind of masonry used in stone arch bridges may be successfully used for the purpose. Ashlar, rubble and boulder masonry facing have all been employed. Plenty of headers should be used, and especial care taken to bind the facing firmly to the concrete backing. Metal clamps are sometimes used to assist in binding the facing to the concrete. The same care should be taken in cutting and setting the arch ring stone as in a masonry arch bridge. The soffit of the arch is never stone faced. Brick facing is sometimes used in place of a stone facing.

Mouldings, Ornamental Shapes and Veneering Slabs.—Mouldings and ornamental shapes are used in various parts of buildings, bridges, etc., for mouldings, corbels, medallions, keystones, railings, posts, etc. These are sometimes cast in place, but where many duplicates are to be used it is most economical to mould them in advance, using the same sets of moulds over and over again. Sand moulds are often used, but the moulds may also be of wood, metal or plaster of Paris.

Casting in Sand Moulds.—The method of casting in sand moulds is similar to that followed in making iron castings. A pattern in wood of the exact size and shape of the desired casting is made, no allowance for shrinkage being necessary. The die is then moulded in iron moulders' sand in a manner entirely similar to that used in preparing the moulds for cast iron, and poured with a concrete mixture of the consistency of cream, composed of cement and finely crushed stone. The excess water soaks into the sand, keeping the concrete moist during setting. The moulds are removed at the end of three or four days and the projecting fins cut off. The sand mould gives a satisfactory surface to the castings. Care should be taken not to use too rich a mortar in making concrete mouldings, as unsightly hair cracks will be likely to form on the surface of the concrete, thereby destroying their beauty and possibly eventually their durability.

Where it is desired to use cement mouldings for various purposes they can be run in long lengths in a sand or metal mould and then cut to the length desired for use. This method is especially to be recommended when the pattern to be moulded repeats a given figure at intervals. Mouldings thus formed are especially adapted for cornices, belt courses and panel facings in buildings, hand rails on bridges, etc.

Wood Moulds.—Wood moulds have been successfully used. There is danger, however, of wood warping and cracking, and care should be taken to keep the surface of the wood well shellaced to prevent moisture from penetrating the wood and causing it to swell and crack or warp.

Cement Moulds.—Cement moulds may be used in many cases for making the plainer ornaments or mouldings. To make a cement mould a reverse impression of the original is obtained by first coating the pattern with linseed oil to keep the cement from sticking, then pouring cement over the original pattern. When the cement has hardened it can be removed and the reverse impression obtained used as a mould for casting the ornament. Before using the cement mould thus obtained it should be given a coat of liquid asphalt, cut with turpentine or benzine. This will give the mould a smooth surface, which, if coated with soap solution, will not stick to the casting.

Stamped Metal and Glue Moulds.—Stamped metal may be used as moulds for ornamental work. The metal may be stamped in one piece or consist of several pieces soldered together. This mould may be used direct, but finer detail will be secured if a glue coating is used to take the impressions. Glue moulds give much finer lines than those obtained by other methods. With a well made glue negative as many as twenty impressions may be obtained, as its elasticity permits the removal of work having considerable undercut. The most intricate designs can be made from glue moulds, and a little practice should enable the workman to turn out very satisfactory ornaments.

The glue mixture for making moulds is prepared as follows: Take the required amount of the very best glue that can be obtained, place it in cold water over night; the next morning, when removed, it will be found to have swollen. The water absorbed will be sufficient to melt it when heated. Mix with this glue an equal amount of glycerine, place the vessel containing them in a

Fig. 58.—Ornamental Work in Moulded Concrete.

steam or hot water bath until the water is nearly all evaporated and until the combined weight of the glue and glycerine about equals the weight of dry glue and glycerine used. This compound of glue and glycerine will never dry, and a mould of it can be melted and used over again many times.

Figure 58 shows two more or less intricate designs of cement or cast stone ornaments which give some idea of the possibilities of cast mouldings. Cast ornaments can be made at a cost far below that of hand-cut stone ornaments.

Concrete Sidewalks.—Concrete, when carefully placed, proves very satisfactory for constructing sidewalks. After the ground has been excavated to the required sub-grade, a sub-foundation usually of broken stone, gravel or cinders is carefully tamped in place. Care should be taken to properly drain the foundation, as if water collects and freezes there is danger of cracking and displacing the surface of the walk. The foundation consists of a layer of 1 : 2 : 4 or 1 : 3 : 6 concrete, 3 or 4 ins. in thickness. Portland cement should be used with stone and gravel less than 1 in. in size, the concrete being mixed medium wet, so that moisture will show on the surface without excessive tamping.

A top surface of cement mortar, usually a 1 : 1 or 1 : 2 mixture, is then spread over the concrete and well worked in to form a wearing surface. Usually a coarse sand or fine gravel is used for the aggregate. When great wear is expected, crushed granite chips or flinty pebbles may be used for the aggregate. Hard, clean sand, however, will usually answer.

Special care is necessary to secure a uniform and evenly graded surface. After the sub-foundation is placed, side pieces to act as forms to retain the concrete are put in place and held from spreading by stakes driven 3 or 4 ft. apart to proper grade. These side pieces act as guides for the straight-edge used in leveling off the concrete and wearing surface.

The sub-foundation should be well sprinkled and the concrete well tamped in place in sections about the width of the walk. A board is placed across the trench to retain the concrete. The concrete may be lined up with a straight-edge, as shown in Fig. 59, leaving from ½ to 1 in. for wearing surface. Three-eighth-inch sand joints should be used to separate adjacent sections, and should not be placed more than 6 or 8 ft. apart. These joints will prevent expansion cracks, or in case of settlement will confine the cracks to these joints. The location of the joints should be marked on the side of the forms, and care taken to form the joints in the wearing surface on the same vertical plane.

The top dressing should follow up closely the concrete work, as it is desirable that the two set together. This top dressing should be worked well over the concrete with a trowel, pressing it heavily onto the concrete surface. Care should be taken that no air spaces are left in the mortar. The leveling off of the surface may be done with a straight-edge. The success or failure of the walk will depend upon the thoroughness with which this work is done, since a good bond between the wearing surface and concrete base is absolutely essential. The mortar will work best when somewhat stiff. As soon as the film of water begins to leave the surface a wooden float is used, followed up by a plasterer's trowel, the operation being similar to that of plastering a wall. The floating should not be continued too long, as it will bring a layer of neat cement to the surface and probably cause the walk to crack. The floating should be done lightly, to compact the surface and give it an unmarked appearance. The surface is then divided into sections over the joints in the concrete. This is done with a trowel, guided by a straight-edge,

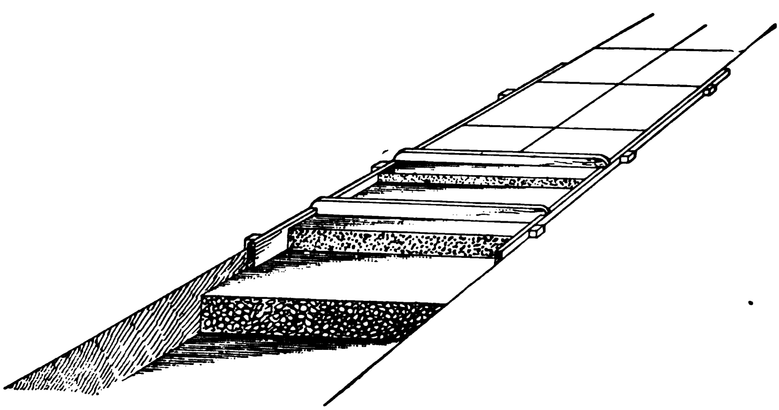


Fig. 59.—Sketch Showing Sidewalk Construction of Concrete.

after which the edges are rounded off with a special tool called a jointer, having a thin, shallow tongue. A special tool called an edger is run around the outside of the walk next to the mould, giving it a neat, rounded edge. A toothed roller, having small projections on its face, is frequently used to produce slight indentations on the surface, adding somewhat to the appearance of the walk. The complete work must be protected from the sun and kept moist by sprinkling for several days.

Figure 60 shows the tools usually employed for finishing sidewalk surfaces and similar cement constructions.

The above method of finishing sidewalk surfaces may be used with slight modifications in finishing almost any kind of surface where a smooth, uniform surface is desired.

Variation in Color of Concrete.—Variation in the color of concrete surfaces may result from one of several causes. A variation in the color of sand used, the presence of dirt or other impurities, may cause a difference in color in different parts of the same wall. A change in the brand or even different batches of the same brand of cement may change the shade, as no two cements are the same in color. A variation in the wetness of the mixture may change the character of the concrete and also its

Fig. 60.—Tools Used in Finishing Cement Surfaces.

coloring. The remedy for any of these causes evidently consists of as careful a selection of materials as is possible, the use of only one brand of cement on a piece of work where a uniform appearance is desirable, and the use of a mixture of uniform consistency throughout the structure. Care should be taken to keep the forms clean, and avoid allowing any dirt to get into the concrete while mixing or depositing, as this will often permanently soil the concrete.

Efflorescence.—Another cause of ugly blotches, consisting of white and yellow stains on the surface of concrete exposed to the action of the weather, is termed efflorescence. The real cause of the deposition of this incrustation is not positively known, but it may probably be explained in one of the following two ways:

First, the failure of the most finely pulverized portion of the cement to be acted upon chemically by the water, the cement remaining inert and afterwards being washed to the surface, where it is deposited and there forms an unsightly incrustation. The incrustation is at first white, and afterwards turns yellow. This action is not unlike that which takes place in concrete deposited under water. As concrete is placed in water a light colored, powdery substance is held in suspension by the water and is usually called "laitance." When a concrete is mixed very wet the same action usually occurs. An analysis of this laitance shows a composition agreeing very closely with that of cement, and it must be inferred that the laitance represents an actual loss of cement.

Second, it has been observed that efflorescence rarely occurs when certain brands of cement are used, and when others are employed it is much more apt to appear. It seems probable that in many cases the trouble is caused by the presence of certain ingredients in the cement, probably sulphates of calcium, magnesium, etc.

These sulphates are soluble in water, and when the wall is soaked they are dissolved and carried to the surface, where they are deposited when the water evaporates. A careful chemical investigation of various cements which do and do not effloresce would doubtless prove of great value in this connection.

Whether the efflorescence is due to one or the other of these causes, the action and the results are the same. If the wall be kept continuously wet the water will finally dissolve out all discoloring matter, and will deposit it on the face of the wall. The rain beating continuously on the face of the wall will gradually dissolve and wash off the incrustation, and after a time the whole discoloration will disappear. However, this action is more or less uncertain. In fact, the efflorescence may appear soon after the wall is built, or it may be that a long period will pass before this action takes place. The bleaching process may be extremely slow, sometimes lasting for years before the discoloration finally disappears, and on this account any attempt to remove it by scraping or dressing the wall will prove futile.

This discoloration is most frequently noticed at and below renewal joints where the laying of the concrete has been stopped perhaps over night. Laitance appears at the surface where the concrete was stopped. A close examination of this surface shows its presence in the form of a very thin layer of a soapy consistency. Where new work is joined to old, there is an excess of cement at the joint which makes it much more waterproof than the body of the wall. Water percolating through the wall washes out the above-named soapy material at this joint, thus causing efflorescence.

Mr. C. H. Cartlidge, M. Am. Soc. C. E., Bridge Engineer, C., B. & Q. R. R., states that the removal of this material by scrubbing the joint with wire brushes and then flushing with water from a hose prevents entirely the appearance of efflorescence at or below renewal joints.

While this method may avail for the removal of efflorescence due to laitance at renewal joints, as has been stated, the efflorescence may be due to the presence throughout the mass of the concrete of uncombined cement or soluble salts, which will be dissolved out and stain the wall; hence we see that other methods of treatment may be necessary.

By the use of one of the methods used to make concrete impervious by the addition of alum and soap to the mixture the efflorescence can be effectually prevented.

Again, if the concrete be laid fairly dry and deposited in layers slightly slanting downward toward the back of the wall, the drainage will be carried away from the face, and all objectionable incrustation be deposited on the back. This may be rendered doubly effective by treating the face of the wall with a wash for rendering it impervious, such as Sylvester Process, page 141.

Again, this process may be applied to walls made of very wet concrete, the wash preventing the escape of the efflorescence to the face of the wall. Other similar methods for rendering the concrete waterproof, which need not be described here, may be used equally well.

The efflorescence may be removed by the use of wire brushes, the sand blast, tool.dressing the surface by hand or with a pneumatic hammer, or by washing it with diluted acids. Washes of diluted hydrochloric, acetic, or oxalic acids may be used for this purpose. A wash consisting of a solution of 1 part of hydrochloric acid and 5 parts of water was successfully used in removing the efflorescence from a reinforced concrete bridge at Washington, D. C. This wash was applied vigorously with scrubbing brushes and immediately washed off with water from a hose to prevent the penetration of the acid. The cost for plain walls was about 20 cents per sq. yd.

But, as already explained, there is no assurance that further efflorescence will not take place, and that such mechanical removal may prove only temporary.

Protecting Concrete Surfaces.—After the concrete has been deposited it is necessary to keep it moist if its surface is exposed to the direct rays of the sun. If the weather is hot and dry, it is also desirable to keep the surface moist, as a certain amount of water is necessary to the process of setting. If this is not done, the concrete will crack and the surface become unsightly, the cracks varying from a hair line to those of considerable size Whenever it is feasible the surface may be sprinkled with water two or three times a day to keep it in condition. Burlap may be spread over the surface and kept wet. This will retain moisture for some time, and thus prevent danger from cracks. Sand and sawdust may be used in the same manner, but are not as effective.

CHAPTER IX.

GENERAL PHYSICAL PROPERTIES.

Retempering.—It is customary in specifications to require that cement or concrete be put in place before a certain interval of time has elapsed, as there is danger of injuring the strength of the concrete if it is disturbed after the initial set has begun. Very few data are available in regard to the effect of disturbing cement or concrete after the period of initial set has begun.

A series of tests was made by Messrs. Goddard and Evans, at Ohio State University, in 1892, to determine the effect of retempering. A batch of mortar was mixed in the morning to about the consistency of mortar as used in practice and was left on a clean glass. At intervals during the day the batch was stirred up and enough water was added each time to make the batch plastic. After eight hours the mortars were placed in the moulds and left over night. Generally the briquettes were sufficiently hardened in the morning to be removed from the moulds. Comparative tests were made on mortars of the same mixture without retempering. The briquettes were broken at 7, 28, 56 and 84 days.

It was found that the Portland cement mortars lost 45 per cent. of their strength at the end of 7 days, 20 per cent. at the end of 28 and 56 days, and 28 per cent. at the end of 84 days, while the Rosendale cements lost 83, 65, 54 and 42 per cent. at the ages of 7, 28, 56 and 84 days, respectively.

A series of tests was made at the Watertown Arsenal to ascertain the effect on the final strength of delaying the time of putting the gauged material into the moulds. Some samples were prepared, tamped into the moulds and allowed to set without being disturbed. At later intervals, other materials were taken from the mixing bed and similarly treated. In one series material was kept in the mixing bed a period of 102 hours before the last sample was drawn. It was found that cementitious properties still remained in the material, as shown by the possession of tensile strength when subsequently tested at the age of one month.

Chief interest, however, is attached to the behavior of materials which were kept in the mixing board for a few hours only, and were used during the day the material was first mixed. The results of some of these representative tests are shown on the diagram (Fig. 61) of cement held in the mixing beds at different periods before the final setting.

A domestic Portland is shown by the lower curve. This was 28 days old at the time of testing; 28.6 per cent. of water was used in the initial gauging. More water was added from time to time as samples were taken out at two-hour intervals, in order to keep the batch plastic. The largest increment of water needed for these periods was added at 10 hours after mixing. The total

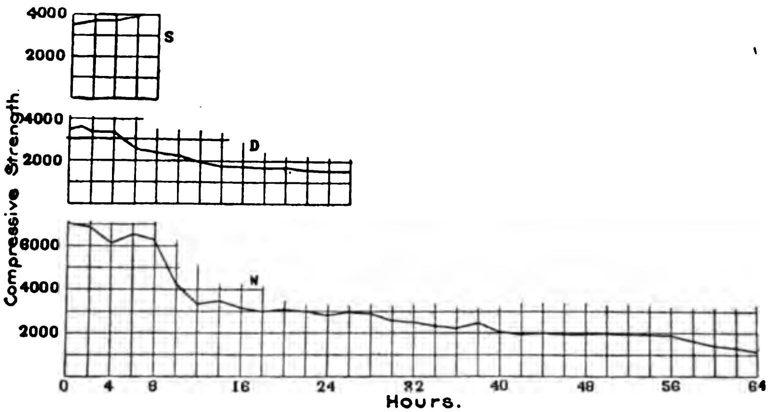


Fig. 61. Diagram Showing the Effect of Retempering Cement Mortars.

quantity of water eventually used was nearly double the original amount.

Tests were made on 4-in. cubes that had reached the age of about one month. Those which were placed in the moulds immediately after gauging had a crushing strength of 7,000 lbs. per sq. in. The strength was well maintained for a period of 8 hours, at which time the strength was still about 6,000 lbs. per sq. in. Cubes at a period of 24 hours had nearly 3,000 lbs. strength, and those after a period of 2½ days displayed a strength of about one-fifth of the original batch.

A German Portland was also used, being kept in the mixing bed a total period of 25 hours. The original gauging strength was about 3,600 lbs. per sq. in. At the end of 8 hours it had a strength of 2,400 lbs. per sq. in.; that is, two-thirds of the original strength remained. At 25 hours, 44 per cent. of the original strength remained.

In another case, not shown on the diagram, a domestic Portland developed practically the same strength in each of the several samples up to the limit of 8 hours' delay; and still another domestic brand showed a higher strength in the 6 and 8-hour samples than those taken out of the mixing bed at earlier hours. There were brands which did not display so favorable results as those described, but in general a considerable part of the strength was retained by samples at a period not over 8 hours.

These results should tend to relieve undue anxiety concerning the necessity of the very early use of cement after gauging. It should, however, be remembered that the mortar must be remixed or broken up thoroughly before it is finally deposited.

The above tests were made under the conditions usual to be met with in a testing laboratory, and should not be taken as giving a criterion of the conditions met with in practical work.

The tendency to use concrete that has been mixed for some time is to place it, after it has stood quiescent on account of unavoidable delays, without remixing and retempering. If the retempering or remixing with an additional amount of water is not done, the mix, if deposited, will be absolutely worthless.

It is interesting to note in connection with the above experiments that the inquiry was extended to embrace material which had hardened for several days and was then broken up and reground to a mortar. A Portland which had become so hard that a pickaxe was needed to break it up was reground and regauged 6 days and 2 hours after the original gauging, and even after so long a time as this it was found that the material acquired strength, and at the age of 1 month had a compressive strength of 700 lbs. per sq. in. The normal strength at this age was approximately 6,000 lbs.

A natural cement was experimented upon which had set sufficiently in four or five months to become resonant. The material for test purposes was scraped off the main batch. That which was scraped off two days after the original gauging, and then made into cubes, had a strength at the end of the month about one-half the normal value. Grouts were made in the afternoon and set the following morning and then retempered. In some cases surplus water was removed. At the end of 30 days the compressive strength ranged from 2,000 to 3,000 lbs. per sq. in.

The lesson to be drawn from these experiments seems to be that the ordinary properties of cement continue active for some time after what is known as the final set has taken place. The loss of strength, as shown by these experiments, due to breaking the materials up, represents the strength gained up to the time of disturbing the mix. The additional or remaining strength shown at the end of the testing period represents the gain in strength from the time of remixing up to the time of final testing.

Table XX. shows the effect of retempering cement mortars in tests made by Philip L. Wormeley, Jr., Testing Engineer, Office of Public Roads (see Farmers' Bulletin No. 235, Department of Agriculture). The mortar used consisted of Portland cement and crushed quartzite. In each case, after the initial or final set had taken place, sufficient water was added in retempering to restore the normal consistency. The briquettes were tested at the age of four months.

TABLE XX.

SHOWING THE EFFECT OF RETEMPERING ON CEMENT MORTARS.

Treatment of mortar.	Tensile strength in pounds per square inch.			
	Neat cement. (a)	1 part cement, 1 part sand. (b)	1 part cement, 2 parts sand. (c)	1 part cement, 3 parts sand. (d)
Mortar made up into briquettes immediately after mixing.	651	624	527	417
	650	701	493	385
	673	624	529	421
	634	581	480	403
	679	610	492	409
Average	657	628	504	407
Mortar allowed to take initial set, then broken up and made into briquettes.	671	692	589	326
	593	670	554	349
	644	654	559	330
	633	676	534	358
	724	700	532	267
Average	653	678	554	326
Mortar allowed to take final set, then broken up and made into briquettes.	455	527	492	364
	522	569	491	380
	525	587	497	361
	558	566	486	315
	642	568	531	345
Average	540	563	499	353

(a) Initial set, 1 hour 42 minutes; final set, 7 hours 15 minutes.
(b) Initial set, 1 hour 30 minutes; final set, 7 hours 15 minutes.
(c) Initial set, 2 hours; final set, 7 hours.
(d) Initial set, 2 hours 20 minutes; final set, 7 hours.

Effect of Freezing on Concrete.—There is considerable difference of opinion among American engineers in regard to whether or not freezing, and even alternate freezing and thawing, will injure Portland cement concrete. Falk states in his "Cements, Mortars and Concretes," "that the hardening properties of frozen cement are not impaired if the freezing has taken place before the initial setting of the cement has begun. Under those conditions the physical action of the changing of the water into globules of ice has prevented the chemical action of the crystallizing of the cement particles; crystallization can not take place until the ice globules return to the liquid form. No damage will then have been done if freezing does not again take place before the cement has set, but if continued thawing and freezing take place, allowing an intermittent action of setting, it is very likely, under those conditions, that the cement will be injured. It is only necessary to bear in mind that the physical action of freezing must so far precede the beginning of the chemical action as to preclude the latter's taking place."

Prof. Spencer B. Newberry states that "freezing does no harm to Portland cement after the mass has fully set. The hardening of the cement is interrupted by freezing, but proceeds again without hindrance after thawing takes place. Damage from frost is to be feared before the setting, especially if excess of water is used. When work in extreme cold cannot be avoided, the sand and water should be warmed and the proportion of water reduced to a minimum. After putting in place, the work should be covered with straw or other non-conductor to protect it from frost. Mortar for use in freezing weather is often made with the addition of salt (about one pound to one gallon of water) and appears to give good results."

Edwin Thacher states that the Melan Arch Bridge at Mishawaka, Ind., having three spans of 110 ft., was built between Oct. 26, 1903, and Feb. 25, 1904, in a temperature ranging from 0 to 55° above. The concrete was heated by admitting hot water to the mixer, and was deposited at about blood heat, and retained enough heat to melt snow at the end of 48 hours. No injury to the concrete could be observed. In this case the mass of concrete was of considerable thickness, and retained heat much longer than it would under ordinary conditions.

There are, however, innumerable examples of concrete which has been seriously injured by freezing. It would seem advisable whenever possible to avoid laying concrete for reinforced structures during freezing weather. When it is necessary to lay concrete in low temperatures, every precaution should be taken to secure the safety of the work, and no loading should be placed upon it until a sufficiently long time has elapsed for it to set after thawing out.

Calcium chloride is also used to lower the freezing point of mortar and concrete. It has a somewhat lower freezing point than salt brine, which is usually assumed to lower the freezing point about $1\frac{1}{2}°$ for each per cent. added. Salt has the effect of slightly reducing the early strength of cement, but probably does not affect the ultimate strength when used in quantities not exceeding 10 per cent. Salt also delays the setting of cement. No tests are available which give the effect of calcium chloride on the strength and activity of cement. However, neither of these ingredients will give the necessary action under low temperatures for 10 per cent. only reduces the freezing point to 17° F. and 20 per cent. to 2° F. Ingredients used to lower the freezing point of concrete should be used with care; in fact, the author does not feel that when other methods can be used that they should be employed when great strength is required as in reinforced concrete.

Impermeable Concrete.—At times an impermeable concrete is desired, as in reservoirs, concrete pipes or to keep dry inclosed spaces below the water level. Numerous experiments have been made to determine the most efficient means of accomplishing this result. From these experiments the following general statements have been deduced: (1) The richest mortar and concrete show the least permeability; (2) when water passes continuously through concrete its permeability decreases very rapidly, and it will generally, after a time, become practically impermeable; (3) the ingredients should be proportioned so as to secure the densest possible concrete with an excess of cement; (4) the mixing should be done with great care to secure a thoroughly homogeneous mass; (5) plenty of water should be used, as wet mixtures do not pass water as readily as dry ones; (6) mixtures from 1 cement and 3 of sand and broken stone to 1 cement and 6 of sand and stone will usually give satisfactory results for moderate pressures. A mixture richer than 1 : 3 is liable to crack or check.

In general it may be stated that in monolithic construction a wet mixture, a rich concrete and an aggregate proportioned to secure great density will in the majority of cases give the desired results.

During the construction of the arched concrete dam at Barossa, South Australia, a number of tests* were made to determine the permeability of concrete under high heads. The aggregate was broken sandstone, 1/8 to 2 in. size, with sizes mixed so as to have 35 per cent. voids. The sand was a mixture of natural sand and stone dust (1/8 in. and less in size) in about equal proportions. The sand used was thoroughly washed. The concrete was mixed by machine in 1/2 cu. yd. batches.

Six cubes, 2 × 2 × 2 ft., were made. In the center of each block a T-piece pipe was embedded and a pressure equal to 100 ft. of water was applied through a 1/2 in. diameter pipe from the top of an adjacent cliff. To prevent the ends of the T-piece becoming blocked with mortar, it was bound around with hemp and small rope to form a bulb about 5 ins. in diameter.

Table XXI. summarizes the results of these percolation tests:

TABLE XXI.

GIVING RESULTS OF PERCOLATION TESTS ON CONCRETE BLOCKS SUBJECTED TO HYDROSTATIC PRESSURE.

	Proportions of Ingredients					Head of Water = 100 ft.		
No.	Cement.	Sand.	Aggregate (1/8" to 2" with varying voids.)	Water used per cu. yd. in mixing, gallons.	Excess mortar %	Age of block when put under pressure.	Time of first appearance of water on surface after turning on pressure, mins.	Measured quantity of water passing through blocks, pints.
1	1	1.84	5.26	28	5	11	Unreliable.	Unreliable.
2	1	1.84	5.26	26	5	11	34	3/4 in 7 weeks.
3	1	1.50	4.63	27	5	10	18	1/30 in 4 weeks.
4	1	2.00	4.50	27	15	10	14	14 in 2 weeks.
5	1	1.75	4.13	28	15	9	12	27 in 7 weeks.
6	1	1.50	4.12	27	10	8	35	1/50 in 2 weeks.
7	1	1.50	3.90	24	12½	6	28	1/8 in 2 weeks.
8	1	1.50	3.70	23	15	5	30	1/50 in 2 weeks.

At the end of 80 weeks the same blocks were subjected to a 200-ft. head, but the percolation was not measured, as in each block the effect closely resembled the results obtained from the head of 100 ft.

*Mr. Alex. B. Moncrieff, M. Am. Soc. C. E., M. Inst. C. E. Trans. Assoc. of C. E., Cornell University. Vol. XIII.

While the above results seem to vary through a great enough range for all practical purposes, even the greatest percolation here shown for 100 ft. head, is negligible.

Rich Surface Coatings.—On horizontal or inclined surfaces, a rich coating of 1 to 1 or 1 to 2 mortar may be used to secure an impervious surface. This should be laid while the concrete is still green and carefully troweled in place. This coating varies from ¼ in. to 1 in. in thickness. There is danger, however, when such a surface coating is used of its cracking and peeling off if exposed to the direct rays of the sun. If it is covered with water, as in the bottom and sides of a reservoir, no danger of this kind should be apprehended.

Alum, Lye and Cement Wash.—A waterproof mixture of alum, lye and cement from which good results have been obtained is made in the following proportions: Dissolve 1 lb. of concentrated lye and 5 lbs. of alum in 2 gallons of water, care being taken to have every particle dissolved. Heating to near the boiling point will quickly insure this without injury to the mixture. This constitutes the stock mixture and may be used in any quantity. To one pint of the stock add 10 lbs. of cement, thinning it with water until the mixture spreads easily and well on the surface to be treated with a calcimine or whitewash brush, filling all the pores. The mixture will be found to be satisfactory when it lathers freely under the brush. Usually one pint of the stock put into a 12-gallon pail and 10 lbs. of cement stirred in, with enough water added to well fill the bucket, makes the wash about right.

Much depends upon the condition of the surfaces to be coated. If they are too dry, wet them down with a brush ahead of the water-proofing, the object being to apply the wash as thin as practicable without running, rubbing it well into the surface with the brush. The wash should be applied while the concrete is still green, or within three or four days from the time it has been laid.

The wash is not found to be satisfactory on old work, and should be applied while the concrete is protected from the sun and while it is still moist; otherwise, too rapid evaporation of the water in the wash will leave the cement without the necessary moisture to set and leave it so that it can be brushed off. The wash should not be applied too thick, as it is liable to scrape off. Where a 1:2 facing mixture is used on the concrete, a 1 part stock to about 30 parts water will give good results. This leaves the surface as it comes from the moulds, without showing the marks of the brush. Mr. U. G. Hayne states that he has seen this wash used successfully on fortification work and for water-proofing tanks, etc. In one case, two tanks made to hold 6 ft. of water are stated to have been in continuous use for over seven years without any loss of water, except by evaporation. Two coats of wash were applied to the inside and bottom of the tanks and also to the outside walls. In this case the concrete was first plastered with a coat of 1 part cement to 2 parts of stock mortar. It is also stated that this wash will prevent fungus growth or discoloration of surfaces covered with it. It may be successfully used for closing the pores of plaster and insure dry walls in building construction.

Prof. Ira O. Baker gives the following formula for making impervious mortar: 1 per cent. by weight of powdered alum is added to the dry cement and sand, and 1 per cent. of potash soap (ordinary soft soap is good) is dissolved in the water used in mixing. The chemical action set up makes an insoluble compound, which practically fills all pores, making an impervious concrete.

Prof. W. K. Hatt, Assoc. M. Am. Soc. C. E., has successfully used a 5 per cent. solution of alum and water and a 7 per cent. solution of soap and water, these solutions being used in equal parts in mixing the concrete. European engineers have extensively used a coating of from ½ to 1 in. of rich cement mortar to secure impermeability.

Sylvester Process of Waterproofing.—If it is desired to render a wall waterproof after construction, it may be treated with waterproofing washes, as in the Sylvester process. This process consists in applying two washes or solutions to the surface of a wall, one composed of castile soap and water, and the other of alum and water. The proportions are three-quarters of a pound of soap to one gallon of water, and half a pound of alum to four gallons of water, both substances to be perfectly dissolved in water before using. The walls should be perfectly clean and dry, and the temperature of the air not below 50° F. when the compositions are applied.

The first, or soap water, should be applied when boiling hot, with a flat brush, taking care not to form a froth on the masonry. This wash should remain 24 hours, so as to become hard and dry before the second or alum wash is applied, which should be done in the same manner as the first. The temperature of the alum wash, when applied, may be 60° or 70° F., and this also should remain 24 hours before applying a second coat of the soap wash and so on. Several coats are necessary to secure an impervious coating, the soap and alum combined forming an insoluble compound, filling the pores of the concrete and preventing the passage of water.

Cost of Sylvester Process.—According to Mr. W. C. Hawley, as given in Gillette's "Cost Data," the cost of Sylvester wash and mortar is as follows: The Apollo Water-Works Co.'s covered concrete water well leaked, and it was therefore plastered with a Sylvester mortar; 1¼ lbs. of a light colored soft soap were dissolved in 15 gallons of water. Three pounds of powdered alum were mixed with each bag of cement. The mortar was 1 : 2. Two coats of this mixture were applied to the walls, giving a thickness of ½ in. This stopped the leaking completely.

The cost was as follows:

2 lbs. soap (with 24 gals. water), at 7½ cts.	$0.15
12 lbs. alum, at 3½ cts.	.42
Total	$0.57

or 57 cts. for soap and alum per bbl. of Portland cement.

A Sylvester wash was used in repairing the bottom of a reservoir lined with 4 to 6 ins. of concrete. The soap solution was composed of ¾ lb. of Olean soap to 1 gal. of water, both being well dissolved and the soap solution boiled. This boiling hot soap solution was applied to the clean dry concrete, 24 hours later the alum wash, 24 hours later the soap wash, and then 24 hours later the alum wash again. Two men applied the solutions, using whitewash brushes, while a third man carried pails of the solution. While making the soap solution, two men attended the 4 kettles, one man kept up the fires and two men carried the solutions to the men applying it. It required fewer men to make the alum solution as it was made cold in barrels. After the second soap wash had been applied to the concrete slope, it became so slippery that the men had to be held by ropes to prevent falling. A rope was placed around two men who started work at the top of the slope, while a third man payed out the rope. The work was done in 8½ days, and cost as follows:

Labor:

1,140 hrs. labor, at 15 cts.	$171.00	
83 hrs. foremen, at 30 cts.	24.90	
83 hrs. waterboy, at 6 cts.	4.98	
Add for supt., 15%	30.13	
Total labor	——	$231.01

Materials:

900 lbs. Olean soap, at 4⅓ cts.	$39.00	
210 lbs. alum, at 3 cts.	6.30	
6 whitewash brushes (10 in.), at $2.25	13.50	
6 stable brushes, at $1.25	7.50	
Total materials	——	$66.30
Total labor and materials		$297.31

This covered 131,634 sq. ft., hence the cost of the two coats of soap and alum was $2.26 per 1,000 sq. ft., or about 0.23 cts. per sq. ft. All but one leak from a small crack were stopped.

The concrete lining of a new reservoir near Wilmerding was waterproofed by using caustic potash and alum in the finishing mortar coat. The stock solution was 2 lbs. of caustic potash and 5 lbs. alum to 10 quarts of water. This was made in barrel lots from which 3 quarts were taken for each batch of finishing mortar, which consisted of 2 bags of cement mixed with 4 bags of sand. A batch of mortar covered an area 6 ft. by 8 ft. by 1 ft. thick. The extra cost of this waterproofing was:

100 lbs. caustic potash, at 10 cts.	$10.00
70 lbs. caustic potash, at 9 cts.	6.30
960 lbs. alum, at 3¼, 3¾ and 4 cts.	34.38
60 hours' mixing, at 15 cts.	9.00
Freight, express and hauling	11.50
Total for 74,800 sq. ft.	$71.18

It will thus be seen that the cost was 95 cts. per 1,000 sq. ft. or less than 0.1 ct. per sq. ft. The cost was less than by using the Sylvester wash, and the result was better, for with the latter the penetration is only 1/16 to ⅛ in. It was found that if less than 2 parts of sand to 1 part of cement was used the mortar cracked in setting. Clean sand is imperative, as any organic matter soon decomposes and leaves soft spots. Do not use an excess of potash; a slight excess of alum, however, does not decrease the strength of the mortar.

Other Waterproof Compounds.—There are a number of compounds on the market of a fatty or waxy nature, which, when mixed with cement to the amount of one or two per cent. of its weight, increase its water-resisting qualities to a marked degree. By a thorough mixing of one to two pounds of a suitable compound to each sack of cement a mortar which is practically waterproof may be obtained at a small additional cost. In purchasing these waterproof compounds, however, care should be taken to select such as have proved to be permanent in effect, as some materials used for this purpose lose their effect after a few days exposure to the weather and are entirely worthless. Data in regard to the permanency and waterproof qualities of these various compounds are not available, and it will be necessary to use much care and judgment in selecting them, otherwise their use will prove a disappointment and a needless outlay of money.

One of the best known waterproofing compounds of this character is manufactured by the Sandusky Portland Cement Co., and is known as the Medusa Waterproof Compound. It is stated by the manufacturers that the addition of this compound, to the amount of 1 to 1½ per cent. of the weight of cement used, is sufficient to render concrete building blocks, made from a 1 cement 5 aggregate sufficiently waterproof to allow plastering direct on the inside surface, and that walls so built are perfectly dry. The compound is prepared by mixing from one to two per cent. of the preparation with the dry cement before adding the sand and water. This preparation is sold in 40 lb. sacks at 12 cts. per pound f.o.b. cars at Sandusky, Ohio. An absorption test was made by the manufacturers as follows: Hollow blocks 8 x 9 x 16 inches made of 1 part cement and 4.8 parts limestone screenings by weight with and without the waterproofing compound were exposed to weather for 9 months, and then allowed to become air dry in the laboratory, then placed in water and weighed at 1, 2, 3, 4 and 24 hours. The results were as follows:

Amount waterproof compound, per cent. of weight of cement.	Weight dry.	Water absorbed, per cent. of total weight. 1 hr.	2 hrs.	3 hrs.	4 hrs.	24 hrs.
No. 1—Waterproof compound 0%	60.62 lbs.	4.38	4.84	4.94	4.99	5.16
No. 2—Waterproof compound 1%	62.34 lbs.	.44	.44	.48	.54	.70
No. 3—Waterproof compound 2%	62.06 lbs.	.09	.09	.14	.14	.30

Waterproof Portland Cement.—It is stated that the Star-Stettin Portland Cement Works have recently invented a process by which a waterproof cement is obtained. The process is secret and details cannot be obtained, but it is probable that the waterproofing qualities of this cement are obtained by mixing some substance with the cement during the process of manufacture. The chemical action taking place on the cement in setting affects the material in such a way that it closes the pores in the concrete without injuring in any way the strength of the cement. It is stated that this preparation will successfully resist the action of frost, heat, hot water, sea water and dilute acids. If this proves to be the case, the problem of securing a waterproof concrete for engineering structures has been solved. Dr. Michaelis recommends the use of barium chloride for this purpose when the concrete is subjected to the action of sea water. Pages 152-154 should be read in this connection.

Asphalt Waterproofing.—Asphalt is extensively used as a waterproofing material. It may be used alone or with paper or felt. When applied to concrete surfaces it will generally be found most satisfactory to first coat the dry surfaces of the concrete with a coat of asphalt cut with naphtha. This is applied as a paint to the concrete when the latter is perfectly dry and then the surface is covered with an asphaltic mastic composed of one part of asphalt to four parts of sand. This is smoothed off with hot smoothing irons and thoroughly tamped and pressed into place. When this coating is applied to the surface of a structure, which is later to be covered with earth or broken stone, it is better to cover the surface of the asphalt with washed roofing gravel so that the broken stone will not cut or damage the asphalt surfaces.

Various methods are employed in waterproofing concrete surfaces with asphalt, such as embedding burlap, felt, paper, or other fabric, in the asphalt coating.

It is very difficult to make hot asphalt adhere to a concrete surface, however dry the same may be, unless it is heated by artificial means. Hot asphalt laid on ordinary dry concrete will not adhere and can be rolled up like a blanket after it is cool. By heating the surface of the concrete with hot sand before applying the asphalt, the latter may be caused to adhere to the same. It will, however, generally be found preferable to use the asphalt cut with naphtha, applying it as a painting or swabbing coat. The cost of this work with the present prices of first-class asphalt will range from 10 to 20 cts. per sq. ft., depending upon open conditions. No special expert knowledge is needed for its application. Some judgment should be used in selecting the asphalt to be used for waterproofing purposes.

The following specifications for asphalt and naphtha waterproofing are used by the Chicago & Northwestern Ry.:*

"The asphalt used shall be of the best grade, free from coal tar or any of its products, and which will not volatilize more than ½ per cent. under a temperature of 300° F. for ten hours. It must not be affected by a 20 per cent. solution of ammonia, a 35 per cent. solution of hydrochloric acid, a 25 per cent. solution of sulphuric acid, nor by a saturated solution of sodium chloride.

"For metallic structures exposed to the direct rays of the sun, the asphalt should not flow under 212° F., and it should not become brittle at 15° F. when spread thin on glass. For structures underground, such as masonry arches, abutments, retaining walls, foundation walls and building subways, etc., a flow point of 185° F. and a brittle point of 0° F. will be required. The asphalt covering must not perceptibly indent when at a temperature of 130° F. under a load at the rate of 15 lbs. per sq. in., and it must remain ductile at a temperature of 15° F. on metal structures, and at 0° F. on masonry structures underground.

"Before applying asphalt to a metal surface, it is imperative that the metal be cleaned of all rust, loose scale and dirt, and if previously coated with oil, this must be burnt off with benzine or other suitable means. The metal surface must be warm to cause the asphalt to stick to it, and the warming is best accomplished by covering it with heated sand, which should be swept back as the hot asphalt is applied. When waterproofing masonry structures, if the surface cannot be made dry and warm it should be first coated with an asphalt paint made of asphalt reduced with naphtha. This is particularly necessary for vertical structures.

"The asphalt should be heated in a suitable kettle to a temperature not exceeding 450° F. If this is exceeded, it may result in 'pitching' the asphalt. Before the 'pitching' point is reached, the vapor from the kettle is of a bluish tinge, which changes to a yellowish tinge after the danger point is past. If this occurs the

*W. H. Finley, M. Am. Soc. C. E. Paper read before Cement Users' Association.

material should be tempered by the addition of fresh asphalt. The asphalt has been cooked sufficiently when a piece of wood can be put in and withdrawn without the asphalt clinging to it.

"The first coat should consist of a thin layer poured from buckets on the prepared surface and thoroughly mopped over. The second coat should consist of a mixture of clean sand or screenings, free from earthy admixtures, previously heated and dried, and asphalt, in the proportion of 1 of asphalt to 3 or 4 of sand or screenings by volume. This is to be thoroughly mixed in the kettle and then spread out on the surface with warm smoothing irons, such as are used in laying asphalt streets. The finishing coat should consist of pure hot asphalt spread thinly and evenly over the entire surface and then sprinkled with washed roofing gravel, torpedo sand or stone screenings to harden the top. The thickness of the coating will depend on the character of the work, and may vary from ¾ in. to 2 ins.

"Where a quantity of asphaltic concrete is required, such as in trough floors on bridges, the concrete should be made in the proportion of 1 part asphalt, 2 parts sand and 3 parts limestone screenings, thoroughly mixed and rammed into place with tamping irons on the first coat of pure asphalt with which the metal was originally covered. At all drainage holes large size stones should be carefully placed by hand to secure perfect drainage."

Asphalt or Felt Waterproofing.—Layers of waterproofing paper or felt cemented together with asphalt, bitumen or tar are extensively used for waterproofing purposes in concrete floors, roofs and walls of underground structures, as tunnels, subways, etc. The materials range from ordinary tar paper laid with coal tar pitch to asbestos or asphalted felt laid in asphalt. Coal tar products will not prove permanent, as they will deteriorate when exposed to moisture. Asphaltic mixtures when exposed to the action of illuminating gas will also deteriorate. In the construction of the New York subway, layers of felt laid on with hot asphalt were used for waterproofing purposes, but trouble was experienced in several stations as the roof was found, after a time, to leak. After a careful investigation it was found that the asphaltic compound had been seriously injured by gas escaping from the mains in the street. Care should be taken under similar conditions to prevent gas from coming in contact with waterproofing mixtures.

Method of Laying Paper or Felt.—It is customary to first place a layer of concrete or some other material upon which to place the waterproofing compound, then mop over the prepared surface with hot asphalt, spread the felt or paper, lapping the latter from 4 to 6 ins. After the first layer of the felt is in place the whole is mopped over with the hot asphalt compound. Another layer of paper is placed and the operation continued until the desired thickness is secured. From two to six layers were used in the construction of New York's rapid transit subway. After this coating is in place the remaining concrete, forming the floor or upper surface, is then laid upon the top layer of the asphalt.

Cost of Waterproofing with Tar Felt and Asphalt.—The cost of waterproofing the New York subway with asphalt felt, and asphalt, as given in Engineering-Contracting, July 18, 1906, is as follows:

	Per sq. yd., single.
1.11 sq. yds. asphalt felt, at 4½ cts. (including 15% for laps)	5 cts.
0.37 gal. asphalt, at 12 cts.	4½ "
Labor	5½ "
Total	15 cts.

This is for one thickness of felt, so that for three thicknesses the cost would be 45 cts. per sq. yd. for labor and materials. Both labor and materials are high in cost. Labor cost was high because of poor supervision. The material was high priced, because asbestos felt dipped in asphalt was specified.

The cost of lining a large asphalt reservoir, as given in Gillette's "Handbook of Cost Data," page 296, is as follows: Cost of first asphalt coat on concrete bottom (34,454 sq. ft.).

	Total Cost.	Cost per sq. ft.
Labor:		
Building sheds, 25 hrs. at 20c.	$5.00	$0.00015
Spreading, 38 " " 20c.	7.60	0.00022
Boiling, 37 " " 15c.	5.55	0.00016
Helpers, 48 " " 15c.	6.45	0.00019
Sweeping, 44 " " 15c.	6.60	0.00019
Materials:		
Asphalt, 18,490 lbs. at $0.01225	226.50	0.00658
Fuel, 1 cord	2.50	0.00012
Hauling 9.25 tons, at $0.47	4.35	0.00007
Totals	$264.55	$0.00768

This gives a cost of $0.06912 per sq. yd.

Cost of second asphalt coat on bottom (34,454 sq ft.).

Labor:	Total Cost.	Cost per sq. ft.
Building sheds	$5.00	$0.00015
Spreading, 35 hrs. at 15c.	5.25	0.00015
Boiling, 30 " " 15c.	4.50	0.00013
Helpers, 52½ " " 15c.	7.88	0.00023
Sweeping, 44½ " " 15c.	6.68	0.00020
Foreman, 17½ " " 25c.	4.38	0.00013
Materials:		
Asphalt, 19,591 lbs., at $0.01225	239.99	0.00702
Fuel, 1 cord, at $2.50	2.50	0.00007
Hauling 9.8 tons, at $0.47....	4.61	0.00013
Totals	$280.79	$0.00821

This gives a cost for the second coat of $0.07389, or a total cost for two coats of .14301 cts.

The waterproofing used for the Atlantic Ave. subway of the Long Island Railroad consisted of a tar felt paper, mopped with a coating of pitch. Over this was spread a 1 in. coat of cement mortar.

The roofing felt consisted of pine wood paper pulp, or asbestos pulp, which had been thoroughly treated and soaked in refined coal tar, and which weighs for single ply at least 15 lbs. per 100 sq. ft. What is known as "medium hard" coal tar pitch of somewhat softer grade than used for roofing purposes, was swabbed over the masonry after it was thoroughly set and dried out. This pitch was poured or mopped onto the concrete surface until it had a perfectly uniform thickness over every part of not less than 1-16 in. The roofing felt was then laid upon the coat of pitch, while it was still soft, the felt being lapped at least 4 ins. on all cross joints, and at least 12 ins. on all longitudinal joints. This was mopped over with a coating of pitch, and upon that a second thickness of roofing felt was placed, and thereupon a third coating of not less than 1-16 in. of coal tar pitch was deposited. The 1 in. coating of 1 to 2 cement mortar was then laid in uniform squares, in every respect similar to the plaster on top of granolithic pavement.

The average labor cost of placing the two layers of felt and three coats of tar pitch was 5½ cts. per sq. yd. The average labor cost of mixing and placing the 1 in. layer of cement mortar was 14½ cts per sq. yd., making a total cost for labor of 20 cts. per sq. yd.

Assuming that the cost of paper felt was 3 cts. per sq. yd. and the coal tar pitch at 12 cts. a gallon, the total cost per square yard for this work was:

	Two layers per sq. yd.
2.13 sq. yds. paper felt, at 3c.	$0.064
0.75 gals. pitch, at 12c.	.09
Labor laying felt	.055
Labor laying cement mortar	.145
Total cost	$0.354

The cost of laying the top finish 1 in. thick of a cement walk, as given by Mr. C. M. Saville, M. Am. Soc. C. E., in Gillette's "Handbook of Cost Data," page 179, is as follows:

	Per cu. yd.	Per sq. yd.
4 bbls. per cu. yd., at $1.53	$6.12	$0.171
0.8 cu. yd. sand, at $1	.80	.018
Lampblack	.29	.009
Labor (2 walk masons, 1 helper)	6.36	.144
	$13.57	$0.342

Thus we see by the above data that the costs of various classes of work vary considerably under different conditions.

The Effect of Sea Water Upon Portland Cement Mortar and Concrete.—The action of sea water upon Portland cement mortars is a phenomenon which is little understood. While it is true that some concrete masonry has withstood the action of sea water for a long time, other structures have been rapidly destroyed when subjected to the same action. Why one structure has resisted well, while another similarly located and perhaps constructed with the same kind of cement has been rapidly destroyed, is a question which has long puzzled chemists and engineers. Many theories in regard to the action of sea water upon the cement have been advanced, and something learned in regard to the subject, but thus far it has not been possible to determine whether or not a structure will stand when subjected to the action of sea water. It has been learned, however, that if any considerable amount of certain ingredients are present, failure is almost certain to take place, while, if they are absent, or only present in small quantities, there is at least a possibility of the structure standing for a time.

The German Portland Cement Manufacturers' Association for a number of years has beeen conducting a study of the action of sea waters upon Portland cement mortars and concrete, and, largely through their efforts, considerable knowledge is available on this subject.

Sea water contains small percentages of magnesium chloride and magnesium sulphate. These two ingredients are supposed to attack the free lime present in the cement, forming calcium-aluminum sulphates. The magnesium chloride has but a feeble action, but the magnesium sulphate attacks the lime with great energy. According to Dr. Michaelis and M. Vicat, the action is explained by the chemical equation: $Ca\ (OH)_2 + MgSO_4 = Mg\ (OH)_2 + CaSO_4$.

The calcium sulphate, owing to its taking a crystalline form with an increase in volume, swells and destroys the mortar.

Alumina and gypsum are also supposed to be injurious. It was discovered by Messrs. Michaelis and Candlot that aluminate of lime, $Al_2\ O_3,\ 3\ CaO$, which is present in the cement, possesses the property of combining with sulphate of lime so as to give the double salt $Al_2\ O_3\ 3\ CaO,\ 3\ (SO_3\ CaO)$ combined with a large quantity of water with great increase in volume. This substance has no firm coherence, and is soluble in pure water, but not in lime water. Whether this action, or the one previously explained, or some other action takes place, we must conclude that cements, rich in lime or alumina, or which contain a high percentage of gypsum, are dangerous for use in sea water, as they all disintegrate rapidly. On the other hand it has been found that the presence of high percentages of silica and ferric oxide seem to be beneficial.

Dr. Michaelis found that the chemical action can be greatly improved by adding some pozzolanic material like trass to the cement. This material combines with the lime, forming a stable compound, which hardens under water. The lime necessary for hardening the pozzolani will come from the cement. This secondary hydraulic action greatly improves the resisting power of the cement. It seems probable that well burnt clay may be used to replace the trass.

It has been found that a dense concrete withstands the action of sea water better than a porous concrete, as the water does not readily penetrate the mass, and chemical action is not so readily set up. Hence well balanced mixtures, as dense as possible, should be used. It has also been found that mortars made with fine sands are much more readily decomposed than those made with coarse sands. Hence fine sands should be avoided for this class of work.

Messrs. Candelot, Le Chatelier, Vicat, Rebuffet and Feret, as well as Dr. Michaelis, have extensively studied the action of sea water on cements. The recommendations of Dr. Michaelis, the greatest living authority on this subject, are given below. The general conclusions to be drawn from the writings of these authorities may be summed up as follows: Sand with a large percentage of fine grains should not be used for mortar and concrete intended for use in sea water. A moderate amount of fine grains, when mixed with coarse and graded sized grains, will, however, increase the density of the concrete and prove beneficial. It also follows that the aggregate should be proportioned to secure the greatest density. Gypsum, which is sometimes used to regulate the time of setting, is dangerous. Portland cement for sea water should be low in alumina (8 per cent. being the maximum amount allowable, and as low as possible in lime). Puzzolanic material is helpful when added to cements to be used in sea water.

Concrete Structures in Sea Water.—On the subject of the permanency of cement concrete when exposed to the action of sea water, Dr. Wilhelm Michaelis says in a paper* on the subject:

"The main points to be considered in erecting permanent structures in sea water with the aid of hydraulic cements, in other words, concrete, are:

(1) From the physical point of view, completely impermeable mixtures should be made, composed of 1 part of cement with 2, or at the most 2½ parts of sand, of mixed grain, of which at least one-third must be very fine sand. To this the requisite quantity of gravel and ballast should be added. This impermeable layer should surround the porous kernel on all sides in sufficient thickness, even underneath. It would, perhaps, be unnecessary waste of material, in the case of thick walls, to use the impermeable mixture throughout; but, so far as possible, the compact shell and the poorer kernel should be made in one operation. Where this is not possible, and the shell is added subsequently, numerous iron ties should be used.

(2) From the chemical point of view, cements or hydraulic

*Trans. Inst. Civ. Engrs., Vol. XVII., p. 375.

limes rich in silica, and as poor as possible in alumina and ferric oxide, should be used, for aluminate and ferrate of lime are not only decomposed and softened rapidly by sea water, but they also give rise to the formation of double compounds, which in their turn destroy the cohesion of the mass by producing cracks, fissures and bulges. The salts contained in sea water, especially the sulphates, are the most dangerous enemies of hydraulic cements. The lime is either dissolved and carried off by the salts, and the mortar thus loosened, or the sulphuric acid forms with it crystalline compounds as basic sulphate of lime, alumina sulphate and ferro sulphate of lime, which are segregated forcibly in the mortar, together with a large quantity of water of crystallization, and a consequent increase in volume results. The separation of hydrate of magnesia is only the visible, but completely innocuous sign of these processes. The magnesia does not in any way enter into an injurious reaction with silica, alumina, or ferro oxide; it is only displaced by the lime from its solution in the shape of a flocculent, slimy hydrate which may be rather useful in stopping the pores, but can never cause any strain or expansion, even if it subsequently absorbed carbonic acid.

The carbonic acid, whether derived from air or water, assists the hydraulic cement as a preservative wherever it comes into contact with the solid mortar. It could only loosen the latter if present in such an excess that bicarbonate of lime might be formed.

(3) The use of substances which render the mortar, at any rate in its external layers, denser and more capable of resistance. Such substances are:

(a) Sesquicarbonate of ammonia (from gas liquor) in all cases where long exposure to the air is impossible. Such a solution, applied with a brush or as a spray, and then allowed to dry, converts the hydrate of lime into carbonate of lime. The latter is not acted upon by the neutral sulphates present in sea water. It must be repeated that it is a decidedly erroneous opinion that the texture of otherwise sound cements is injured by the action of carbonic acid; on the contrary, it renders them more capable of resistance, except in the above mentioned case, which is extremely rare, when bicarbonate of lime is formed and goes into the solution.

(b) Fluosilicates, among which magnesium fluosilicate is most to be recommended. The free lime is converted into calcium fluoide and silicate of lime, and, in conjunction with the liberated hydrate of magnesia, these new products close the pores of the mortar. Both salts are sufficiently cheap to be used on a large scale.

(c) Last, not least, barium chloride. Two or three per cent. of the weight of the cement is dissolved in the water with which the concrete is mixed. This forms perfectly insoluble barium sulphate with the sulphates of the sea water, while the magnesia remains in the solution as magnesium chloride. Although in this case there can be no further closing of the pores, yet the insoluble barium sulphate, which is formed, affords some protection and does not give rise to any increase of volume (swelling). From two to three per cent. of barium chloride does not in any way diminish the strength, as has been proved by means of comparative tests of English and German cements. Frequently the strength of the mortar is increased by this addition. This substance is only to be used in the external, perfectly water tight skin of the concrete; in other words, in the 4 to 8 in. coating, composed of 1 cement, 1 to 2 sand and 3 to 4 gravel, flint, broken stone, etc."

Strength of Cement Mixtures in Sea Water.—It has been found that the strength of a cement mixture does not increase as rapidly in salt water as in fresh. Tests made by the Boston Rapid Transit Commission show that, during the early stages of setting, fresh and salt water briquettes possess practically the same strength, but at 9 months sea water briquettes decreased considerably in strength.

According to M. Feret,* tension specimens hardened in sea water are stronger than those hardened in fresh water, but with compression specimens the reverse is true. In conclusion, it may be stated that if the material does not fail by disintegration, its strength under sea water will approximate but never exceed that of materials setting in fresh water.

The Effect of Oil on Cement and Concrete.—Until within the last two or three years there has been considerable difference of opinion in regard to the effect of oil on concrete. Even to-day many consider oil entirely harmless, and to prove their assertions

*Proceedings of Institute of Civil Engineers, Vol. CVII., p. 163.

call attention to machinery foundations in use for many years, which, though exposed to waste oil, are perfectly sound. On the contrary there are many who believe that concrete is injured by oils and substantiate their beliefs by citing examples of concrete which disintegrated when oil appeared to be the sole cause.

In 1903 it was accidentally discovered in the cement testing laboratory of the Chicago, Milwaukee & St. Paul Ry. that oil disintegrates Portland cement. A neat Portland cement briquette, two years old, which had been used in the laboratory as a paper weight, was laid aside, where it was exposed to occasional droppings of signal oil. In ten months the briquette began to disintegrate. This led to an extensive investigation as to the effects of oils on cement and concrete by the engineering department. The following are the results of the tests, taken from an article* by James C. Hain, Assoc. M. Am. Soc. C. E., then Engineer of Masonry Construction C., M. & St. P. Ry. An examination was made of a great many concrete structures on which more or less oil was found. There were a limited number of instances where the concrete was possibly affected by oil. In these cases, however, the concrete was very old, and the character of the original material and the workmanship were questionable. On the contrary no concrete which was built in late years, and known to be of good quality, was affected to any perceptible degree. One case which particularly attracted attention was the concrete floor of an oil house in which lubricating and lighting oils had been stored for six years without any apparent effect. The penetration of the oil was slight, perhaps not to exceed 1-16 in. Moreover, the saturated portion seemed to be as sound as the rest. There were other cases where the oil had penetrated deeper. For example, in the pits of the round house the oil had gone in from ¼ to ½ in. In other respects the concrete seemed perfectly natural. These pits, however, had been in use only about a year when investigated. These observations proved nothing definite and, while the investigation seemed favorable for the structures examined, it could not be taken as conclusive for all concrete structures.

Laboratory experiments were then taken up on Portland cement briquettes made of neat cement, of 1 : 3 sand mortar and

*Eng. News, March 16, 1905.

of 1 : 3 mortar of limestone screenings, which were allowed to age four days in the laboratory air, and were then subjected to applications of signal oil. At first small quantities of oil (enough to saturate) were applied daily. Later the applications were less frequent, depending upon the amount of oil absorbed. Cracks developed in the sand and limestone briquettes first at the age of 2½ months, while the neat briquettes showed cracks at the end of five months. All the briquettes ultimately disintegrated. The cement used in these tests was what is known as a stone and clay cement; 18 briquettes each of neat cement of 1 : 3 sand and of 1 : 3 limestone screenings gave practically uniform results as to time of disintegration, as given above.

After the results of these preliminary experiments were available a more extensive series was started. Instead of confining the tests to a single kind of cement and to signal oil, as was done in the first series, three kinds of cement and characteristic oils or fats of five different groups were employed. The oils and fats used were as follows:

Class....	Animal fat	Animal oil	Vegetable oil		Mineral oil
			Semi-Drying	Drying	Crude
Kind....	Ext. of lard	Whale oil	Castor oil	Boiled Linseed oil	Petroleum oil

Cylinder oil, which is a mixture of animal fat and mineral oil, was also used for the purpose of comparison. Well known brands of cement were used, one being selected from cement made from stone and slag, another from marl and clay and the third from slag and limestone. Neat briquettes and 1 : 3 sand briquettes for all varieties of cement were treated with each of the six varieties of fats and oils. All briquettes were left in the laboratory air seven days before starting the oil treatment. The oil applications were continued for nine months after they were started. A summary of the results is given in Table XXII.

The greatest effect was caused by animal fat or extract of lard oil. It disintegrated most of the neat and sand briquettes in from 2 weeks to 2½ months, although it failed to destroy some even at the end of nine months. As a rule the neat briquettes were destroyed first, which was contrary to what might be expected. It was found that in general cement made from stone and clay were affected the least, while slag cements were affected the most; this, however, was not true in all cases, and the peculiar characteristics of each cement and not the materials from which it is made affected it the most.

Next in effect was signal oil, a mixture of animal fat and mineral oil. It acted only slightly different from extract of lard. Following this were the whale and castor oils, which caused much less disintegration than either of the two just mentioned, affecting but small percentage of the briquettes. Petroleum and boiled linseed oils did not disintegrate any of the briquettes up to nine months. Petroleum, however, penetrated and affected the strength somewhat, and possibly would have eventually destroyed it, while boiled linseed oil formed a coating without penetrating. Of the five classes, boiled linseed oil was the only one that apparently did not affect the strength of the briquettes. This was probably due to the oxidation, which prevented it from soaking in.

Further tests were made on older briquettes cured according to the regular laboratory practice. Some of these tests were as follows: A neat, a 1 : 1, a 1 : 2, and a 1 : 3 sand briquette, all of which were two years old, were dried at the stove for 20 days and then treated with signal oil. After two years, with one exception, they showed no signs of disintegration. The above briquettes were made of silica cement instead of regular Portland, which consisted of equal portions of sand and Portland cement, ground together to a fineness that passed through a No. 200 sieve. This cement makes weaker briquettes than those made from standard Portland cement. The one that failed was the weakest of the four, being a 1 to 3 mixture. Again a neat and a 1 : 3 sand briquette were taken from the vat at the age of one year and treated with signal oil. They appeared to be perfectly sound after being soaked about one year. A 28-day briquette, after being dried in the laboratory air for three months, was treated with oil and was not distintegrated until after eight months. Eight-year-old briquettes were also treated and were unaffected after nine months. A piece of concrete from the oil house floor spoken of above was immersed in oil for ten months and was unaffected.

While the tests on comparatively new briquettes showed with but one or two exceptions disintegration from the action of oil, the tests on old specimens showed up much better. Out of 15 old briquettes, seasoned according to the usual laboratory practice, only two failed under the action of oils, although treated from nine months to 2 years, the two that failed being the weakest. The briquettes which were unaffected were cured from one to two years in water.

TABLE XXII.

SHOWING THE EFFECT OF OIL ON CEMENT AND MORTAR BRIQUETTES.

No. briquettes made.	Class of Portland Cement.	Mixture Portland Cement and Sand.	Extract Tar Oil.	Whale Oil.	Castor Oil.	Linseed Oil.	Petroleum Oil (crude).	Signal Oil.
			Time applied before disintegration					
18	Stone and clay	Neat	3 mos.	*	*	*	*	*
12	Stone and clay	1:3 sand	*	*	*	*	*	*
18	Marl and clay	Neat	$2\frac{2}{3}$ mos.	*	*	*	*	6 mos.
12	Marl and clay	1:3 sand	*	*	*	*	*	*
18	Slag and stone	Neat	1 mo.	3 mos.	4 mos.	*	*	$1\frac{1}{4}$ mos.
12	Slag and stone	1:3 sand	7 mos.	$4\frac{1}{4}$ mos.	$6\frac{1}{2}$ mos.	*	*	4 mos.

* Sound after applying oil 9 months, at which tests were discontinued.
All briquettes set 7 days in air before applying oil.

It was observed (1) that most oils penetrate concrete mortar and may weaken them. (2) That concrete is more liable to be disintegrated when saturated with oils and fats if not thoroughly set. (3) A good quality of concrete is less susceptible to the effects of oil than a poor quality, such as a porous, frosted, lean, poorly mixed or improperly seasoned concrete. Ordinary concrete work is rarely subjected to continued large doses of oils, being usually only spotted. Under such conditions little danger may be apprehended, especially if the concrete is of good quality and well seasoned, and even if the conditions are more severe there will, generally speaking, be small danger of dangerous disintegration. Experiments were also made for the purpose of determining a cheap manner of treating concrete to prevent the disintegrating action of oil, but with unsatisfactory results, no satisfactory wash being found.

One of the briquettes treated with signal oil was sent to the laboratory of Toch Brothers, Long Island City, and a careful analysis was made of it. Mr. Maximilian Toch states that a determination of the soluble substances in the briquette showed that the disintegration was due to the formation of oleate and stearate of calcium. To reduce this to its simplest expression, the animal cils contain acids which combine with the lime and crystals of stearate and oleate of lime are formed. It is very likely that these crystals in the process of formation have increased the bulk in the briquette and the bond which has been formed by the lime in the set cement has been totally disintegrated and ruptured. These crystals were isolated and verified under the microscope

Mr. Toch also states that machine oils are almost all paraffine oils, do not contain animal fats, and hence do not affect concrete.

Silicate of magnesia, sold under the name of fluate, has often been used as a wash to protect concrete against the action of oil. When this wash is applied to concrete, silicia is liberated and fills up the pores. The magnesium fluate acts as a binder, and the cement becomes excessively hard after a few months. Limestone and building stone have been treated with this material in Europe with great success. This compound is, however, expensive.

Preservation of Metal in Concrete.—One of the most important questions asked in regard to reinforced concrete is: Will it be permanent; will the imbedded metal be preserved from oxidation? If not, the construction will deteriorate. Much evidence has been published showing that iron and steel are perfectly preserved in concrete, but occasionally evidence to the contrary is made known. Prof. Spencer B. Newberry, Assoc. M. Am. Soc. C. E., states the theory of the protection of iron from rust when embedded in concrete as follows:

"The rusting of iron consists in oxidation of the metal to the condition of hydrated oxide. It does not take place at ordinary temperatures in dry air or in moist air free from carbonic acid. The combined action of moisture and carbonic acid is necessary. Ferrous carbonate is first formed; this is at once oxidized tc ferric oxide and the liberated carbon dioxide acts on a fresh portion of metal. Once started, the corrosion proceeds rapidly, perhaps on account of galvanic action between the oxide and the metal. Water holding carbonic acid in solution, even if free from oxygen, acts as an acid and rapidly attacks iron. In lime water or soda solution the metal remains bright. The action of cement in preventing rust is now apparent. Portland cement contains about 63 per cent. lime. By the action of water it is converted into a crystalline mass of hydrated calcium silicate and calcium hydrate. In hardening, it rapidly absorbs carbonic acid and becomes coated on the surface with a film of carbonate. Cement mortar thus acts as an efficient protector of iron, and captures and imprisons every carbonic acid molecule that threatens to attack the metal. The action is, therefore, not due to the exclusion of air, and even though the concrete be porous, and not in contact with the metal at all points, it will still filter out and neutralize the acid and prevent its corrosive effect."

In regard to the action of cinder concrete, Prof. Newberry writes as follows:

"The fear has sometimes been expressed that cinder concrete would prove injurious to iron on account of the sulphur contained in the cinders. The amount of this sulphur is, however, extremely small. Not finding any definite figures on this point, I determined the sulphur contained in an average sample of cinders from Pittsburg coal. The coal in its run state contains a rather high percentage of sulphur, about 15 per cent. The cinders proved to contain 0.61 per cent sulphur. This amount is quite insignificant, and even if all oxidized to sulphuric acid it would at once be taken up and neutralized in the concrete by the cement present and would by no possibility attack the iron."

Prof. Newberry states that a reinforced concrete water main taken up after fifteen years' use in damp ground at Grenoble, France, showed the metal absolutely free from rust and the adhesion perfect.

Mr. E. L. Ransome states that embedded steel rods used in a sidewalk in Bowling Green Park, New York, were found to be in perfect condition after twenty years' use. He also mentions, among other examples, sidewalk slabs in Chicago, which, after being broken up, showed the rods in perfect condition. The slabs were of limestone concrete, and had been in use for eight or ten years. Numerous other examples might be cited.

Mr. Jas. S. Mack, Supt. of the Standard Mines of the H. C. Frick Coke Co., states that a 24-in. cast-iron pipe was used as a discharge pipe to carry off sulphurous water from the mine. This pipe was lined with a coating of Portland cement to protect it from the action of acid in the water. The coat was put on with a brush on the perpendicular line and had a thickness of about ¼ in. On the horizontal line it was put on with a trowel and had a thickness of about ½ in. With the exception of one or two places where the cement had worn through, the pipe remained uninjured after eighteen years of constant service.

Mr. Ernest McCullough states that he placed some badly rusted iron rods in blocks of concrete of a 1:2:4 mixture. The blocks were broken after about a year and the rods were found to be comparatively clean and bright. The rust was gone, but was not adhering in scales or flakes to the concrete. It seemed to have entirely disappeared, leaving the enveloping concrete somewhat discolored. The adhesion of the rods was seemingly perfect.

Mr. Edwin Thacher, after extensive study and observation, states that he considers concrete to be a perfect protection for embedded steel.

Prof. Charles L. Norton, of the Massachusetts Institute of Technology, Boston, Mass., made a large number of experiments with bricks of concrete 3 × 3 × 8 in., in which steel rods, sheet steel and expanded metal were embedded. One portion of the specimens, together with unprotected steel, was enclosed in steel boxes and exposed for three weeks to the action of steam, air and carbon dioxide; another portion to air and carbon dioxide; a third to air and steam, and a fourth left on the table of the testing room. His conclusions were as follows:

First—Neat cement is a perfect protection.

Second—Concrete should be dense, without voids or cracks, and be mixed quite wet when applied to metal.

Third—The corrosion found in cinder concrete is mainly due to iron oxide in the cinders and not to sulphur.

Fourth—Cinder concrete, if free from voids and well rammed when wet, is about as effective as stone concrete in protecting steel.

Fifth—It is important that the steel be clean when embedded in the concrete.

Sixth—It is essential that the steel be coated with cement before embedding in concrete, the unprotected pieces being found to consist of more rust than steel.

Additional tests made by Prof. Norton were as follows: Specimens of steel, clean, and in all stages of corrosion, were embedded in stone and cinder concrete, both wet and dry mixtures being used, and exposed to moisture, carbon dioxide and sulphurous gases. Some of the samples were treated in tanks supplied intermittently with steam, hot water, moist air, dry air and continuously with carbon dioxide for from one to three months. Under these conditions, unprotected steel vanished into streaks of rust; but, when protected by an inch or more of sound concrete, the steel was absolutely unchanged. He concludes that steel embedded in concrete mixed wet, whether stone or cinder concrete, will be perfectly protected for all time.

M. Breuillié is said to have found that a chemical union takes place between the metal and the cement, forming a silicate of iron which is soluble in water. If this is true, when this salt is dissolved the bond between the metal and the concrete will be destroyed. The many excellent examples of successful reinforced concrete whose strength is dependent upon the adhesion between the two materials would seem to refute this statement. Again, the successful use of cement paint for protection would indicate that this celebrated French engineer is in error in regard to his deductions, or, what is more probable, the cement used by him in his experiments may have contained some injurious agent.

Adhesion Between Concrete and Steel.—It is important that there be a positive bond between the concrete and the steel of a reinforced member. Usually the entire stress in the steel must be transmitted by this bond or adhesion. The bond may be due (1) to the adhesion of the concrete to the steel, (2) to surface friction, probably due to shrinkage strains set up by the concrete upon setting, causing it to grip the steel firmly and generate a high frictional value; or (3) to some mechanical arrangement consisting of a deformed, twisted or corrugated form of rod, giving an effective mechanical bond between the steel and concrete. When the shear per foot run between the steel and concrete exceeds the safe working adhesion or surface friction between the two materials, some form of mechanical bond should be used.

The values given by different experiments for the adhesion in pounds per square inch of contact surface vary quite widely. It is probable that the adhesive strength under normal conditions is great enough to care for the shearing stresses until the elastic limit of the metal is passed, when the bar stretches, decreases in cross-section and is torn from the concrete.

A rough surface gives a higher adhesive value than a smooth surface, rusted bars considerably higher values than those not rusted, while oiling or painting greatly reduces the adhesion. Round bars show the greatest adhesion; flat bars the least.

M. Bauschinger and M. de Joly, from a series of experiments, conclude that the adhesion of concrete to iron or steel rods is from 570 to 710 lbs. per sq. in. of surface. These values appear to be somewhat high after a careful examination of a large number of recent experiments, some of which are here given.

Prof. W. K. Hatt, in the Journal of American Society for Testing Materials, 1902, gives the following values for the adhesion of round rods, each value being the average of three tests, the concrete being a 1 : 2 : 4 mixture, and its age about 32 days:

Size of Rod.	Depth of Rod in Concrete in ins.	Ultimate adhesion in lbs. per sq. in. of rod surface.
7/16 in.	6.0	636
⅝ in.	6.4	756

The following values, Table XXIII., of the holding power of different types of rods, are from a series of tests by Prof. Charles Spofford, Massachusetts Institute of Technology, and reported by him in Beton und Eisen, Part III., 1903. The mixture used was a 1 : 3 : 6 Portland cement concrete, the stone being trap rock. The rods were all thoroughly cleaned with the sand blast before concrete specimens were made. The concrete was sufficiently wet to flush water to the surface when tamped into the moulds. The time of test was 28 days. In the following table each value given is an average of several tests.

TABLE XXIII.

Type of Rod.	Cross-section of Rods used in inches.	Number of Rods used to obtain average value.	Adhesion in lbs. per sq. in. of contact surface.
Ransome	½ x ½ — ¾ x ¾ and 1⅛ x 1⅛	12	296
Thacher	½ — ¾ and 1⅛	9	275
Johnson......	½ x ½, ¾ x ¾ and 1¼ x 1¼	9	339
Plain	¾ in. round	3	245
Plain	¾ x ¾	3	279
Plain Flats...	1⅛ x ½, 1½ x ⅜ and 2¼ x ¼	9	164

The cleaning of the rods by sand blast made them much smoother than they are ordinarily. Hence the values for plain rods given above are, when compared with the values for deformed rods, proportionately less than they would be under ordinary conditions.

The comparative resisting power of twisted and corrugated bars to longitudinal slipping is shown in Table XXIV.

The area of contact surface between the twisted bars and the cement was taken to be the periphery of the bars, multiplied by its length in the prism. In computing the contact surface of the corrugated bars, the periphery was assumed to be represented by the square circumscribing the corrugations. A part of the prism thus enclosed would be subjected to shearing stresses.

TABLE XXIV.—ADHESIVE RESISTANCE OF CORRUGATED AND TWISTED STEEL BARS EMBEDDED IN CEMENT AND MORTAR PRISMS.

(From Watertown Arsenal Tests, 1904.)

Nominal dimensions of prisms inches.......................... 6 x 6 x 12.

Effective sizes of rods taken, inches .. .. {twisted, .81 x .81 / corrugated, .73 x .73}

Atlas brand cement used.

Composition of Prisms.	Kind of Rod.	Ultimate resistance per sq. in.	Remarks.
Cement, neat..................	Twisted	1,278	Rod broke
" "	"	1,303	"
" "	Corrugated	968	"
" "	"	958	"
" "	"	960	"
1 Cement, 1 sand..............	Twisted	1,318	"
1 " 1 "	Corrugated	977	Rod pulled out
1 " 2 "	Twisted	1,199	" broke
1 " 2 "	Corrugated	934	" pulled out
1 " 3 "	Twisted	701	" "
1 " 3 "	Corrugated	735	" "
1 " 4 "	Twisted	796	" "
1 " 4 "	Corrugated	564	" "

The following tests, Table XXV., were made at Columbia University in 1903, and have not been published heretofore:

TABLE XXV.

Type of Rod.	Condition.	Size in inches.	Age of Test.	Adhesion, lbs. per sq. in. sur.	Remarks.
Plain	Rusted	$\frac{3}{4} \times \frac{3}{4}$	1 month	437	Pulled out.
"	"	$\frac{3}{4} \times \frac{3}{4}$	3 months	642	Block split.
"	Clean	$\frac{7}{8} \times \frac{7}{8}$	1 month	294	Pulled out.
"	"	$\frac{7}{8} \times \frac{7}{8}$	3 months	431	Block split.
"	"	$\frac{7}{8} \times \frac{7}{8}$	1 mo. in water	146	Pulled out.
Ransome....	Clean	$\frac{3}{4} \times \frac{3}{4}$	25 days	500	Block split.
"	"	$\frac{3}{4} \times \frac{3}{4}$	"	520	"
"	"	$\frac{3}{4} \times \frac{3}{4}$	1 month	457	"
"	"	$\frac{3}{4} \times \frac{3}{4}$	"	560	"
Thacher	"	$\frac{7}{8}$	"	700	"
"	"	$\frac{7}{8}$	"	788	"
"	"	$\frac{7}{8}$	"	450	"
"	"	$\frac{7}{8}$	"	410	"

According to the above tests, rusted rods give about 50 per cent. higher adhesive values than clean rods.

Prof. Arthur N. Talbot, in the Bulletin of University of Illinois, Sept. 1, 1904, reports the following results (Table XXVI.) of tests of bond between steel and concrete. Plain round and square bars and Johnson corrugated bars were used. It will be seen that the Johnson bars split the concrete, while the plain bars slipped. No slipping could be detected before the maximum load was reached.

The range of resistance per square inch of embedded bars was great, being from 298 to 639 lbs. for the Johnson bar, and from 174 to 360 lbs. for the plain rods. In no case did the tension in the plain rods exceed the elastic limit. In tests Nos. 21 and 22 the bars were placed within 1½ ins. of the face of the concrete block. Nos. 16 and 17, which had 24 ins. of rod embedded, show a small resistance per square inch of surface. This is probably

TABLE XXVI.—BOND BETWEEN STEEL AND CONCRETE.

Test No.	Type of Rod.	Maximum Load.	Area sq. in.	Lbs. per sq. in. of net section.	Lbs. per sq. in of net surface.	Elastic limit.	Remarks.
1	½-in. Johnson	14,990	.20	74,950	625	60,000	Concrete split.
2	½-in. Johnson	14,210	.20	71,050	593	60,000	Concrete split.
3	½-in. Johnson*	12,605	.20	63,000	525	60,000	Concrete split.
27	½-in. Johnson*	15,335	.20	76,650	639	60,000	Cylinder broke.
4	¾-in. Johnson	17,175	.365	47,050	573	58,300	Concrete split.
30	¾-in. Johnson	11,755	.365	32,200	392	58,300	Concrete split.
26	¾-in. Johnson	13,975	.365	38,300	466	58,300	Concrete split.
5	¾-in. Johnson*	16,360	.365	44,800	545	58,300	Concrete split.
31	¾-in. Johnson*	9.515	.365	26,050	317	58,300	Concrete split.
32	¾-in. Johnson*	8,960	.365	24,500	298	58,300	Concrete split.
29	¾-in. Johnson*	10,435	.365	28,600	348	58,300	Concrete split.
33	⅜-in. square	4,780	.16	29,900	250	45,000	Rod slipped.
34	⅜-in. square	6,850	.16	42,800	357	45,000	Rod slipped.
13	⅜-in. square	5,850	.16	36,550	305	45,000	Rod slipped.
35	⅜-in. square*	6,810	.16	42,600	357	45,000	Rod slipped.
36	⅜-in. square*	6,910	.16	43,200	360	45,000	Rod slipped.
18	⅜-in. square*	4,100	.16	25,600	214	45,000	Rod slipped.
14	⅜-in. square*	5,560	.16	34,700	290	45,000	Rod slipped.
8	¾-in. square	11,600	.56	20,620	322	35,000	Rod slipped.
9	¾-in. square	11,850	.56	21,100	329	35,000	Rod slipped.
10	½-in. square	7,910	.27	29,320	317	33,300	Rod slipped.
15	½-in. square	6,400	.27	23,700	256	33,300	Rod slipped.
11	⅜-in. round	3,255	.11	28,800	228	42,500	Rod slipped.
12	⅜-in. round	3,860	.11	34,200	270	42,500	Rod slipped.
16	⅜-in. square†	6,905	.16	43,150	180	45,000	Rod slipped.
17	⅜-in. square†	6,690	.16	41,800	174	45,000	Rod slipped.
21	⅜-in. square	4,785	.16	29,930	249	45,000	Rod slipped.
22	⅜-in. square	6,000	.16	37,500	312	45,000	Rod slipped.
23	⅜-in. square	4,580	.16	28,640	239	45,000	Rod slipped.
28	⅜-in. square	6,540	.16	40,800	340	45,000	Rod slipped.
7	¾-in. round	7,000	.452	15,500	245	40,500	Rod slipped.
6	¾-in. round	11,000	.452	27,500	386	40,500	Rod slipped.

due to uneven distribution of the transmission of stress from the bar to the concrete throughout the length, due to the greater stretching of the bar within the concrete. Age of test was 60 days.

Eliminating tests 16 and 17 and tests struck by sledge, and averaging the values of tests on round bars, we obtain a mean value

*Struck 6 quarter-swing blows with a 10-lb. sledge.
†Embedded for a length of 24 ins.

of 281 lbs., and for square bars a value of 298 lbs., while the average for the Johnson bars is 530 lbs. per superficial inch of contact. The average of 30-day tests at Columbia University on Ransome bars is 509 lbs., and of Thacher bars 587 lbs., while the average of plain bars for one and three months agrees sufficiently well with those given by Prof. Talbot.

We may then conclude that the ultimate surface bonding for plain bars, round and square in cross-section, may be taken at from 250 to 400 lbs. for an age of 30 to 60 days, with an average value of about 300 lbs., and for deformed rods, such as Ransome, Thacher and Johnson, from 300 to 800 lbs., with an average value of about 500 lbs.

The safe working value for adhesion or surface bonding may be taken at from 40 to 100 lbs. per sq. in. for plain rods and from 50 to 150 lbs. for deformed rods.

Shrinkage and Expansion of Cement Mortar and Concrete When Setting.—It has been found that cement mixtures hardening in air shrink somewhat during the early periods of setting, while those hardened in water expand in like manner but in a less degree. The contractions and expansions are greatest in neat cement mortars, while the variations in volume are less in mortars containing sand and in concrete. Prof. G. F. Swain, who made some elaborate experiments upon 5-in. cement mortar cubes in the Massachusetts Institute of Technology laboratories, reports in the Transactions of the American Society of Civil Engineers for July, 1887, that the contraction at the end of 12 weeks is:

For neat cement	0.14% to 0.32%
For 1 cement to 1 sand	0.08% to 0.17%

and the expansion at the end of 12 weeks is:

For neat cement	0.04% to 0.25%
For 1 cement to 1 sand	0.00% to 0.08%

Prof. Bauschinger, of Munich, reports results similar to those of Prof. Swain. His test specimens were cubes 4.72 ins. on a side. The following table shows his results:

Mixture. Cement to Sand.	Age.	Contraction in per cent. Hardening in air.	Expansion in per cent. Hardening under water.
Neat	16 weeks	0.12% to 0.34%	0.01% to 0.15%
1:3	16 weeks	0.08% to 0.15%	.0% to 0.02%
1:3	16 weeks	0.08% to 0.14%	— 0.03% to 0.02%

Mr. John Grant records in Vol. 62, Proc. Inst. Civ. Engr., the results of his experiments on prisms 4 ins. long, hardened only in water. Neat cements at the end of one year expand 0.09 to 0.21 per cent., and a 1 : 3 sand mortar 0.01 to 0.06 per cent. He states that the addition of gypsum increases the amount of the expansion.

In his book on "Portland Cement," Dr. C. Schumann* reports the following results of experiments on prisms 3.9 ins. long and with a cross section of .775 sq. in. These were immersed in water:

Age in Weeks.	Neat Specimen.	1 Cement, 3 Normal Sand.
1	.048%	.015%
4	.082%	.021%
13	.104%	.024%
26	.125%	.028%
39	.139%	.030%
52	.146%	.033%

M. Considère made valuable experiments on the behavior of both plain and reinforced concrete pieces setting in both air and water. The measurements were made with extremely delicate instruments. The mortar used was approximately in the proportions of 1 part Portland cement to 3 parts silicious sand. The

TABLE XXVII.—CONSIDÈRE'S TESTS, SHOWING EXPANSION AND CONTRACTION OF CEMENT PRISMS.

	Contraction of Prisms Setting in air.				Expansion of Prisms Setting in water.			
No. days.	Neat cement, plain.	Neat cement, reinforced.	Mortar, plain.	Mortar reinforced.	Neat cement, plain.	Neat cement. reinforced.	Mortar, plain.	Mortar, reinforced
1	.060%	.006%	.022%	.004%	.007%	.002%	.003%	.002%
2	.058	.009	.021	.006	.015	.003	.010	.002
3	.057	.012	.020	.007	.021	.004	.013	.002
4	.058	.014	.021	.008	.027	.005	.015	.003
5	.060	.016	.022	.009	.032	.006	.017	.003
6	.064	.017	.026	.009	.037	.008	.018	.003
7	.070	.020	.029	.009	.041	.009	.019	.004
14	.095	.022	.038	.009	.059	.013	.020	.004
21	.110	.023	.042	.010	.069	.016	.022	.004
28	.118	.024	.044	.010	.073	.018	.024	.004
35	.123	.025	.045	.010	.075	.020	.026	.005
42	.128	.025	.047	.010	.077	.021	.027	.005
49	.130	.025	.047	.010	.078	.022	.027	.005
56	.131	.025	.049	.010	.078	.022	.027	.005
63	.132	.025	.050	.010	.079	.022	.028	.006

size of the prisms was 2.360 × 0.98 × 23.6 ins. in length. The reinforcement consisted of an iron rod 0.4 in. in diameter. This gives 5.45 per cent. reinforcement. The results are shown in Table XXVII.

At the end of two or three years it was found that the maximum

*"Portland Cement," 1899. by Dr. C. Schumann.

contraction for neat cement not reinforced varied from 0.15 per cent. to 0.2 per cent., and the expansion appeared to be about the same, while the expansion of the mortar appeared to be about one-third of that of the neat cement. A calculation to determine the stresses in the reinforcement of the neat cement prism setting in air gave a mean compressive stress of 7,110 lbs. per sq. in., and a mean tensile strength in the cement of 410 lbs. per sq. in. In the mortar prism the reinforcement had a mean compressive strength of 2,845 lbs. per sq. in., while the tensile stress in the concrete was 155 lbs. per sq. in.

For the prism setting in water the mean tensile strength developed in the reinforcement was about 6,250 lbs. per sq. in., and the mean compressive stress in the cement was about 360 lbs. per sq. in. For the mortar prism, the tensile stress in the reinforcement due to the elongation was about 1,700 lbs. per sq. in., while the mean compressive stress in the concrete was about 100 lbs. per sq. in. These stresses in both cases were for 5.45 per cent. of reinforcement.

M. Considère concludes that the initial tensions developed in the concrete of a prism while setting in air by the presence of reinforcement of sufficient sectional area very nearly approximates the ultimate resistance of similar pieces of plain concrete at the same age, and this is the reason for the regular contraction of reinforced prisms.

A test was made by Mr. H. S. McCurdy for the Boston Transit Commission* to determine the amount of shrinkage of concrete in setting. The specifications for the Boston Transit work called for a 1 : 2½ : 4 concrete, gravel being used for the aggregate. Two beams, one in air and one in water, 8 ins. square, and having an effective length of 8.9 ft. were tested. One end of the beam was anchored to the masonry of the subway, the other end was so connected to the trunnions of a transit instrument that any change in length caused the line of collimation of the telescope to revolve about its axis. The instrument was directed toward a leveling rod 240 ft. away; thus any change of length of the beam was magnified 3,850 times. Some changes in temperature took place during the period of observation and an allowance was made for expansion of .0008 in. for each degree of Fahrenheit. Mr. McCurdy concludes from his observation that

*Seventh Annual Report Boston Transit Commission, 1901.

a concrete beam 100 ft. long, setting in air, would in 12 weeks, if the temperature remained constant, shrink about .028 ft. His other observations were made on a beam of the same size as the first, but which was kept under water twelve weeks. He found that the shrinkage in this, after making allowances for changes in temperature, was about two-thirds that of the beam that was kept in air.

These results do not agree very well with results obtained by other experimenters, but it should be remembered that the experiments were made on concrete blocks, and not mortar specimens as used by others. On the other hand, too few tests were made to draw any definite conclusions.

Coefficient of Expansion.—The coefficient of expansion of concrete, due to temperature changes, does not differ materially from that of steel. In Les Annales des Ponts et Chaussées, 1863, Bonniceau gives the following values per degree Fahrenheit:

Neat Portland cement	.00000594
One cement to two silicious sand	.00000655
Concrete (proportions not given)	.00000795

Christophe, in "Le Béton Armé," quoting Bonniceau, Meies, Adie and Durand-Claye, states that the coefficient varies from .00000667 to .00000805.

To determine the coefficient of expansion of concrete Sir Alexander R. Binnie, M. Inst. C. E., gives the details of his investigation as follows*:

A block of 1 cement, 4 crushed granite concrete, 1 ft. square and 100 ft. long was constructed a few years ago. The block was built to rest on rollers so as to be free to expand in any direction. Proper verniers were attached to either end, moving against pillars detached from the block itself. For three years, summer and winter, expansion and contraction of the block was measured. The expansion and contraction was found to be influenced considerably by the condition of the atmosphere, as well as by the temperature. In wet weather the expansion, due to absorption of moisture, was often as much as that due to summer heat. The action of the sun shining on one side of the block also had a disturbing influence. Taking the average of all conditions in various states of the weather, the expansion was 0.005226 in. for a rise of 1° F. for a block 100 ft. in length

*Institute of Civil Engineers, December, 1904.

and 1 ft. square in cross section. This gives a coefficient of expansion of .000004355.

In this connection it should be remembered that the concrete was a rich mixture and the aggregate granite, which is seldom used for concrete. A series of tests to determine the coefficient of expansion of concrete was made by Prof. Wm. D. Pence of Purdue University. Two series of tests were made. In the first the bars were 6 x 6 x 24 ins., but owing to the great length of time required to heat the 36 sq. ins. of section, bars 4 ins. in diameter and 36 ins. long were used for the second test. The stone was hand broken Bedford öolitic limestone for the first series and Kankakee limestone, crusher broken, for the second tests. A local pit gravel was used in both tests. The coefficient was determined by comparing the expansion of the concrete bar with that of steel and copper bars, subjected to the same condition as regards heat and cold. The reliability of tests thus made involving the comparison of metal and concrete is open to question. The results of these tests are given in Tables XXVIII. and XXIX.

TABLE XXVIII.—COEFFICIENT OF EXPANSION OF 1 : 2 : 4 BROKEN STONE (PORTLAND) CONCRETE.

Series.	Tests.	Kind of Stone.	Brand of Cement.	Standard Bar.	Coefficient of Expansion (F)
1st	No. 5	Bedford	Lehigh	Steel	0.0000052
1st	No. 6	Bedford	Lehigh	Steel	0.0000053
1st	No. 7	Bedford	Lehigh	Steel	0.0000053
1st	No. 10	Bedford	Lehigh	Steel	0.0000057
	Average results of first series................				0.0000054
2d	No. 2	Kankakee	Medusa	Steel	0.0000056
2d	No. 3	Kankakee	Medusa	Copper	0.0000054
2d	No. 8	Kankakee	Medusa	Steel	0.0000057
	Average results of second series.............				0.0000056
	Average of entire series of results on broken stone concrete..........................				0.0000055
	Coefficient of expansion, Kankakee limestone bar				0.0000056

TABLE XXIX.—COEFFICIENT OF EXPANSION OF 1 : 2 : 4 AND 1 : 5 GRAVEL CONCRETE.

Series.	Tests.	Proportions.	Brand of Cement.	Standard Bar.	Coefficient of Expansion (F).
1st	No. 4	1 :2 :4	Lehigh	Steel	0.0000054
2d	No. 4	1 :5	Medusa	Steel	0.0000055
2d	No. 7	1 :5	Medusa	Copper	0.0000053
2d	No. 10	1 :5	Medusa	Steel	0.0000052
	Average of results of second series..........				0.0000053
	Average of entire series on gravel concrete....				0.0000054

Experiments made by Rae and Dougherty, under the direction of Prof. Hallock, Columbia University, resulted in obtaining the following values:

1 cement 2 sand 5 gravel	0.00000655	1 cement 2 sand	0.00000561

An average of these is 0.00000608.
Clark gives the value at 0.00000795.

If an average of the mean values as given by the last three experimenters be taken, there will result the value 0.00000648.

The rate of expansion per degree of Fahrenheit for wrought iron and steel, as given by Kent, is from 0.00000648 to .00000686; and, as given by U. S. Government Reports on Iron and Steel, it varies from .00000617 to 0.00000676. The mean of these values is 0.00000654 or 0.6 per cent. greater than the mean value for the expansion of concrete given above; and, if the mean of the U. S. Government values be used, the difference is 0.2 per cent. greater for the concrete. These values are so nearly equal that it is evident no special consideration need be taken in regard to the relative coefficients of expansion of steel and concrete in structures subjected to ordinary temperatures.

It will be of interest in this connection to compare the coefficient of expansion of concrete with that of various stones and other substances. Table XXX. is taken from Engineering News, Oct. 23, 1902, p. 341:

The method of treating concrete constructions to prevent unsightly cracks, due to expansion and contraction, is discussed under "Retaining Walls."

Fire Resisting Qualities of Reinforced Concrete.—Many claims, some often extravagant, have been made as to the fire resisting qualities of concrete, both plain and reinforced. The word "fireproof" is a relative term. A material that will resist fire at high temperature indefinitely is unknown. Material that will resist the flames and heat of an ordinary conflagration, in such a manner that the structure will be intact and safe after moderate repairs, may be called fireproof. Reinforced concrete certainly falls within this classification, as shown by numerous fire and water tests, and by a number of structures, which have passed through severe fires.

A fire in a building filled with combustible materials will develop, for short periods, temperatures as high as 2,000° F., or higher. The Building Code of the City of New York requires that a structure to be considered fireproof shall withstand, when fully loaded, a temperature averaging 1,700° F. for four hours, and then be subjected to a stream of water discharged from a 1⅛ in. nozzle under 60 lbs. pressure for five minutes without failing. A number of systems of reinforced concrete have with-

TABLE XXX.—COEFFICIENTS OF EXPANSION OF VARIOUS MATERIALS.

Material	Authority.	C Coefficient of expansion.	E Modulus of elasticity.	F for 100° F
Brick, Common........	Haswell	.0000012	3,500,000	420
Brick, Fire	Haswell	.0000028	....	....
Cement	Haswell	.0000080	....	....
Concrete, 1 : 2 : 4—				
Lehigh, Portland and Limestone.........	Pence	.0000055	1,000,000	555
			2,000,000	1,100
Lehigh, Portland and Gravel............	Pence	.0000054	....	....
Granite—				
Aberdeen	Dana	.0000044	....	....
New England.......	Dana	.0000048	5,500,000	2,640
			13,000,000	6,240
Iron or steel, average..	Haswell	.0000066	27,000,000	17,620
Limestone—				
Sing Sing, N.Y.....	Dana	.0000057	....	....
Bedford, Ill.........	Pence	.0000056	1,500,000	940
			3,300,000	1,850
Marble	Ganot and Haswell	.0000048	2,500,000	1,200
Sicily	Dana	.0000061	....	....
Pottery—				
Wedgwood..........	Enc. Brit.	.0000049	....	....
Bayeux	Enc. Brit.	.0000092	....	....
Quartz along axis....	Enc. Brit.	.0000028	....	....
Perp. to axis........	Enc. Brit.	.0000043	....	....
Sandstone	Haswell	.0000068	....	....
Sandstone	Haswell	.0000108	....	....
Sandstone	Ganot	.0000065	....	....
Sandstone, red	Dana	.0000095	....	....
Sandstone, red	Thurston	.0000033	....	....
Sandstone, red	Thurston	.0000055	....	....
Slate	Spon's Dic.	.0000055	7,000,000	3,850
Wood, White Pine.....	Spon's Dic.	.0000023	1,800,000	410

stood this test for beams, floor slabs and columns. Other systems have failed, but the causes contributing to their failure have usually been traceable to concrete not sufficiently dried out, to insufficient thickness of concrete over the metal, and in one case to the use of broken stone containing a high percentage of lime. There is every reason to believe that reinforced concrete, when made of the proper materials should prove a satisfactory fire-resisting material. The relative resisting qualities of concrete made from cinder, stone (trap rock) and slag are in the order named.

Portland cement in hardening absorbs 10 to 20% of its weight of water. This water is chemically combined and none of it is given off until a temperature of at least 500° F. is reached, the dehydration, probably not ceasing until a temperature of about 1,000° F. is reached. This water, as it is given off, vaporizes, and keeps the surrounding materials at a comparatively low temperature. After the dehydration has taken place the concrete is much improved as a non-conductor of heat and greatly retards the dehydration of the adjacent concrete.

Concrete itself is a poor conductor of heat, and the materials a fraction of an inch away may be practically insulated from the action of heat for a long time. Tests show that a thickness of from ¾ to 1 in. of stone concrete is sufficient to protect the metal in a floor slab; and for cinder concrete ½ to ¾ in. is sufficient, while for beams and columns the thickness should be from 1½ to 2 ins. for a stone concrete, and from 1 to 1½ ins. for cinder concrete, depending upon the size of the member. Care must be taken in selecting the cinders for cinder concrete, for if any unburnt coal is present the fire resisting quality is greatly reduced. Limestone should be avoided and granite may chip under the action of heat.

The reinforced concrete factory of the Pacific Coast Borax Co., at Bayonne, N. J., passed through a severe fire in 1902. This structure was 4 stories in height, and, excepting the roof, was built entirely of reinforced concrete. The walls, posts, girders, floors and a number of partitions were of reinforced concrete. The floor slabs were 4 ins. thick, and were supported by beams 24 ins. deep, 4½ ins. wide and 24 ft. long, spaced 3 ft. centers. The columns were square, and reinforced with 4 twisted steel rods tied together at intervals. The walls were 16 ins. thick with 9-in. hollow spaces in the center. The concrete was made from Atlas Portland cement and crushed trap rock, crusher run, all passing through a 1-in. screen. No sand was used, the stone dust taking its place; 1 : 5 and 1 : 6½ mixtures were used.

The contents of the building were entirely destroyed. The walls and floors remained intact, and, except in one place, where an 18-ton tank fell from the roof to the floor, cracking some of the floor beams, and in another place on the outside of the wall, the concrete work was practically uninjured. It is stated that the fire was so hot that it melted brass and iron castings. This would require a temperature of upward of 2,000° F.

The Baltimore fire is often cited as affording examples of reinforced concrete structures, which withstood a conflagration. While much has been written about this fire, the fact remains that the number of reinforced concrete structures that went through it were so few that when the surrounding conditions are considered it does not appear safe to draw any definite conclusions from them.

Very few experimental data are available as to the ability of concrete to withstand cracking or disintegration when subjected to great heat or as to its heat conductively.

Prof. Ira H. Woolson of Columbia University (during the past two years) has made a series of tests on the fire resistance of concrete, that were fully reported in papers read before the American Society for Testing Materials at Atlantic City, N. J., in June, 1905, and June, 1906. An independent investigation was also made by a committee of the National Fire Protection Association, and a report of the result of their tests was presented at the annual meeting, May 23-25, 1905.

Prof. Woolson's Tests.—Fire resistance and crushing tests were made on 4-in. cubes, and tests for elastic deformation and crushing strength were made on 6 x 6 x 14-in. prisms of a 1:2:4 mixture of cement, sand and 3/4 in. broken stone. This is a mixture commonly used for reinforced concrete construction. The cement used was a mixture of different brands of the best grades of American Portland cement. The sand was taken from a quantity used in the construction of a building on Columbia University ground. It was medium size, fine quality and not especially clean. Three varieties of stone, limestone, trap rock and clean 1/2-in. quartz gravel, were used; cinder concrete specimens were also used in the final series. The concrete was mixed moderately wet and well tamped.

The purpose of the investigation was threefold: To ascertain first, at what temperature the concrete began to lose crushing strength due to the action of heat; second, the rate of loss of strength resulting from the increase of heat, and third, the effect of varying temperatures upon the elastic properties of concrete, the purpose being to determine if the elasticity began to diminish prior to the loss of strength or concurrently with it.

The initial tests were made on specimens heated to 500° F., and the temperature was increased 250° F. for each succeeding set, up to 2,250° F. for the final set. In the final tests, made in 1906, it was decided that instead of raising the specimens very slowly up to a furnace temperature of 1,500° to 2,000° F., as done in the initial tests, it would be best to raise the temperature rapidly to some fixed point, then hold it there for a definite period; by these means duplication of tests could be more easily made and the conditions would more nearly approximate those of an actual fire. A temperature of 1,500° F. was adopted as a fair average and the furnace raised to this

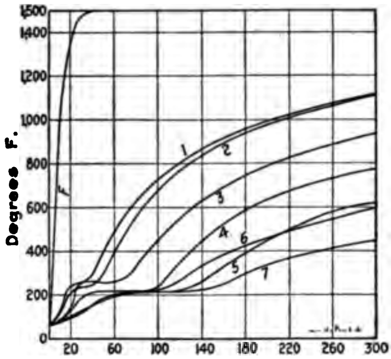


Fig. 62.—Diagram Showing Rise of Temperature in Furnace-Heated Concrete Blocks.

temperature in 40 to 60 minutes and held there until the conclusion of the tests. The heating was done in an oven type of gas furnace. The temperature was measured continuously by a Le Chatelier pyrometer. In the 1906 tests a number of specimens had thermo couples cast in them, but unfortunately, with one exception, they were displaced by tamping the concrete, and the registered temperatures were confusing and of little value.

The gravel specimens were the only ones which attained an interior temperature equal to the furnace temperature. It is rather surprising to note that the cinder concrete specimens came next to the gravel in the amount of interior heat recorded, for cinder concrete is well known to be an effective fire resistant. The thermal curves shown in Fig. 62 will serve to illustrate the rise of temperature in the interior of a specimen. The location of the thermo couples from 1 to 7 ins. from the hot face are shown in Fig. 63. In this particular specimen the position of the thermo couples was absolutely assured, and the resulting curves shown in Fig. 62 can be assumed to be reasonably correct.

It will be observed that the curves all flatten out at or a little after the 212° point is reached, showing that after steam begins to generate there is little if any rise of temperature until all the water in the concrete has been evaporated. While all the curves lag at the 212° point the lag is greater with increased thickness of the concrete. It should also be noted that up to 3 ins. thickness the recorded temperature at which it occurs is

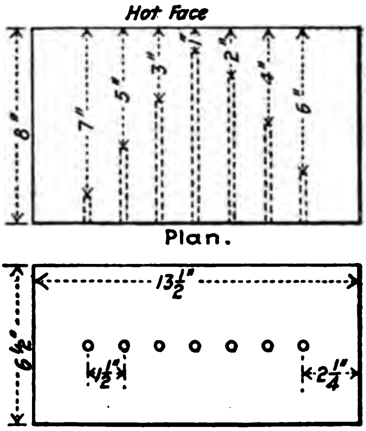


Fig. 63.—Diagram Showing Location of Thermo-Couples in Test Cubes.

above 212°, showing that the surface of the concrete heated rapidly and went above the boiling point of water. It is undoubtedly these conditions which cause steam to generate rapidly enough to become explosive and burst off patches of concrete, which is a common occurrence in fire tests.

The curves 6 and 7 are somewhat irregular. The reason for this is that the points 6 and 7 were 2¼ ins. from the ends of the blocks and received more or less heat through the ends of the block, throwing their temperature curves out of harmony with those of the other points, which were not thus affected.

In the initial tests made in 1905, a test was also made to determine the thermal conductivity of concrete. It was found that by allowing 1 hour and 15 minutes to bring the furnace temperature up to 750° F. and then holding the temperature constant, it required 2 hours and 40 minutes more for the interior of two different prisms to attain the same temperature. Then raising the furnace to 1,000° F. in 30 minutes it required 1 hour and 10 minutes more for the prisms to become uniformly heated throughout. These tests were also made by embedding thermo-couples in the middle of the prisms and connecting them by a switch to the same galvanometer on which the couple in the furnace was recording. The concrete when this test was made was 28 days old. In this instance it required 5 hours and 35 minutes to obtain a temperature of 1,000° F. through 3 ins. of concrete, when the specimens were surrounded by heat on all sides, with no radiation possible.

These two experiments show conclusively the low thermal conductivity of concrete. They also show that two or three inches of concrete properly mixed, tamped and set will resist a fierce conflagration for hours without permitting a serious temperature rise upon the opposite side.

It will be interesting to compare the results obtained by the committee of the National Fire Protection Association with the tests just given. Briefly the tests were as follows: Three round steel rods placed respectively 1 in., 2 ins. and 3 ins. from the bottom were embedded in 8 × 11¾ ins. × 6 ft. concrete beams. Holes were cored in the beams reaching from the top down to the reinforcing rods; in these were placed thermometers. The several beams were laid close together side by side to form the top of a four walled gas furnace, the temperature inside of which was gradually raised during three hours to between 1,900° and 2,000° F. The time required to heat up the rods in all samples, which had only 1 in. of concrete covering to a temperature of 770° F., was well within 1½ hours or an average of 59 minutes for 11 samples. For 2 ins. covering it required 2 hours and 20 minutes and for 3 ins. of material it required an average of 2 hours and 30 minutes. At a temperature of 770° the strength of steel is reduced about 25 per cent.

It is stated that the samples composed of the richest concrete mixtures proved to be the slowest conductors of heat, and that the rise of temperature in the cinder concrete samples was quite noticeably slower than in any of the other samples.

None of the samples showed any signs of breaking or chipping off of the concrete during the fire, but after being removed it was found that the material had lost practically all its strength to a depth of about 4 ins. from the sides and bottom and that it had softened perceptibly clear to the top. The effect was practically similar in all specimens; those containing the most cement

TABLE XXXI.—SHOWING COMPRESSIVE STRENGTH OF 4-IN. TRAP-ROCK CONCRETE CUBES.
(Woolson's Tests.)

Specimen No.	Age in days. Before heating.	Age in days. Between heating and testing.	Heated to degrees F.	Ultimate strength, in lbs. per sq. in.	Average strength, in lbs. per sq. in.	Condition after heating.
1	32	..	Unh't'd.	1,903		
2	32	..	"	1,913	1,903	
3	32	..	"	1,892		
4	36	2	500	1,808		
5	36	2	500	2,100	1,920	
6	36	2	500	1,853		
7	36	2	750	1,880		
8	36	2	750	1,690	1,840	Slight cracks.
9	36	2	750	1,950		
10	36	2	1,000	1,547		Brittle and full minute cracks.
11	36	2	1,000	1,273	1,413	Same.
12	36	2	1,000	1,418		Same.
13	36	2	1,250	1,110		Brittle. Had several small cracks.
14	36	2	1,250	1,163	1,244	Same.
15	36	2	1,250	1,459		Same.
16	50	10	1,500	1,265		Few cracks. Appeared sound.
17	50	10	1,500	1,802	1,556	Sound. No cracks.
18	50	10	1,500	1,602		Same.
19	50	10	1,750	644		Full of cracks.
20	50	10	1,750	1,220	923	Same. One crack entirely around.
21	50	10	1,750	904		Full of cracks.
22	44	9	2,000	680		Full of cracks. One around three sides.
23	44	9	2,000	1,072	847	Few cracks. Surface pitted.
24	44	9	2,000	790		Same.
26	44	9	2,250	626	501	Very much fused on bottom.
25	44	9	2,250	458		Slightly fused on one edge. Few cracks.
27	44	9	2,250	420		Full of cracks. Slightly fused on one side.

appeared, however, to be the most sound. The samples tested were 1 : 2 : 3, 1 : 2½ : 5 and 1 : 3½ : 7 mixtures, the stone being coarse screened gravel, limestone and red granite, not larger than 1¼ ins., and cinders. The cinder mixtures were, however, 1 : 2 : 5 and 1 : 2 : 6. The specimens were from 45 to 48 days old when tested.

Strength Tests.—Tests were made upon specimens both before and after heating to determine the effect of the application of heat upon the specimen. Table XXXI. gives the ultimate crushing strength of the 4-in. trap cubes which were heated to various temperatures and crushed after cooling. No appreciable effect upon the strength can be noted until a temperature of 750° F. is reached; when slightly lower average strengths were obtained.

TABLE XXXII.—SHOWING COMPRESSIVE STRENGTH OF 4-IN. LIMESTONE CONCRETE CUBES.
(Woolson's Tests.)

Specimen No.	Age in days. Before heating.	Age in days. Between heating and testing.	Heated to degrees F.	Ultimate strength, in lbs. per sq. in.	Average ultimate strength in lbs. per sq. in.	Condition after heating.
1	34	..	Unh't'd.	1,968		
2	34	..	"	1,843	1,817	
3	34	..	"	1,640		
4	38	3	500	1,227		Somewhat brittle.
5	38	3	500	1,290	1,234	Same.
6	38	3	500	1,184		Same.
7	38	3	750	1,122		Brittle. Gave metallic sound when struck.
8	38	3	750	1,440	1,244	Same.
9	38	3	750	1,170		Same.
10	38	3	1,000	923		Stone slightly calcined.
11	38	3	1,000	991	1,052	Same.
12	38	3	1,000	1,241		Same.
13	38	3	1,250	988		Calcination throughout.
14	38	3	1,250	1,038	976	Same, but appeared sound.
15	38	3	1,250	903		Same; surface discolored.
16	44	3	1,500	680		Same; edges chipped.
17	44	3	1,500	778	765	Same. Full of small cracks.
18	44	3	1,500	838		Same. Crumbled easily.
19	44	3	1,750	832		Same and discolored.
20	44	3	1,750	684	813	Very fragile.
21	44	3	1,750	922		
22	44	3	2,000	....		Crumbled on cooling.
23	44	3	2,000	....	....	Same.
24	44	3	2,000	....		Same.
25	44	3	2,500	....		Same.
26	44	3	2,500	....	....	Same.
27	44	3	2,500	....		Same.

Beyond 750° F. the decrease was marked, with two or three exceptions, notably at 1,500° F., where two of the specimens gave remarkably high results. The surface of all specimens heated over 750° F. were covered with minute cracks; at 2,250° F. the cubes were slightly fused, due to the fact that fire brick protection was displaced in removing previous specimens, exposing the remainder more or less to direct contact with the flames.

TABLE XXXIII.—SHOWING COMPRESSIVE STRENGTH AND MODULUS OF ELASTICITY OF TRAP-ROCK CONCRETE PRISMS.

(Woolson's Tests.)

Specimen No.	Age in days: Before heating	Age in days: Between heating and testing.	Heated to degrees Fahr.	Ultimate strength in lbs. per sq. in.	E at 200 lbs. per sq. in.	E at 600 lbs. per sq. in.	E at 1000 lbs. per sq. in.	Condition after heating.
2	33	..	....	1,560	3,450,000	2,140,000	1,700,000	Specimen in good condition.
3	33	..	....	1,820	3,180,000	3,000,000	2,440,000	Same.
4	34	..	....	1,725	3,340,000	2,070,000	1,610,000	Same.
10A	48	6	500	1,404	715,000	902,000	863,000	Same.
11A	48	6	500	1,970	834,000	950,000	1,040,000	Same.
12B	49	3	750	1,250	490,000	526,000	472,000	Very minute cracks apparent.
13B	49	3	750	835	400,000	461,000	....	Appeared sound; but brittle.
16C	50	5	1,000	735	128,000	171,500	....	Same. Had metallic ring when struck.
17C	50	5	1,000	1,100	160,000	212,000	....	Same.
20D	54	3	1,250	910	89,000	122,000	....	Surface covered with small cracks.
21D	54	3	1,250	1,055	83,000	125,000	....	Same.
24E	56	9	1,500	250	19,400	....	....	Very bad specimen; sides warped and shattered.
25E	56	9	1,500	....	....	....	....	Worse than 24E.

TABLE XXXIV.—SHOWING COMPRESSIVE STRENGTH AND MODULUS OF ELASTICITY OF LIMESTONE CONCRETE PRISMS.

(Woolson's Tests.)

Specimen No.	Age in days: Before heating	Age in days: Between heating and testing.	Heated to degrees Fahr.	Ultimate strength in lbs. per sq. in.	E at 200 lbs. per sq. in.	E at 600 lbs. per sq. in.	E at 1000 lbs. per sq. in.	Condition after heating.
5	30	..	....	1,427	3,000,000	1,715,000	1,028,000	
6	30	..	....	1,452	3,340,000	2,330,000	2,080,000	
7	30	..	....	1,246	2,500,000	1,715,000	1,390,000	
8A	44	4	500	1,568	700,000	352,000	476,000	Specimen in good condition.
9A	44	4	500	1,207	1,330,000	1,176,000	972,000	Same.
14B	45	7	750	1,110	500,000	333,000	344,000	Not smooth on sides.
15B	45	7	750	1,214	222,000	294,000	286,000	Good condition.
18C	51	4	1,000	932	157,000	200,000	....	Stone on edge slightly calcined.
19C	51	4	1,000	1,145	172,000	285,000	....	Same.
22D	57	3	1,250	840	92,500	13,650	....	Stone entirely calcined to depth of 2 ins.
23D	57	3	1,250	740	59,000	10,000	....	Same.
24E	57	19	1,500	....	....	....	....	Stone entirely calcined; sides warped and shattered.
25E	57	19	1,500	810	83,300	133,000	....	Same as 24E to lesser degree.

Table XXXII. gives similar data for limestone cubes. The strength of the limestone cubes approximated that of the trap mixtures. Heating to 500° F., however, gave a great loss of strength. There were no evidences in the appearances of the cubes to indicate the deterioration. No further weakness resulted at a temperature of 750° F., but beyond this the loss of strength continued. After heating to 2,000° F. and 2,250° F. the cubes appeared strong and in good condition while hot, but when cold they began to disintegrate, and at the end of a few days were so badly decomposed as to be unfit for testing.

Table XXXIII. shows the results of the tests upon trap rock prisms. The values for the modulus of elasticity for the unheated specimens approximate closely results obtained by other investigators. As usual, the value of E diminishes with increase of pressure. With the heated specimens this is not so marked; in fact, it is often the reverse, particularly with the intermediate loadings. The value of E, however, decreases rapidly, due to the heating. This change is very apparent, even with a temperature of 500° F., and the value gradually decreases with the increase of heat.

Table XXXIV. gives the data for limestone prisms. The value of E falls rapidly with the increase of heat applied, the same as for the trap rock mixture. The surfaces of the prisms of both mixtures were covered with minute cracks after being subjected to over 750° F. and then cooled. These cracks increased in number and size as the heat became higher, and at 1,500° the prisms began to warp and disintegrate on cooling. This deterioration increased with time, and at the end of 9 days one prism of each mixture was so badly crumbled it was unfit for testing. This disintegrating effect is probably due to the swelling of the cement as a result of recalcination.

The great loss in strength and elasticity of concrete when subjected to severe and continued application of heat would lead to the conclusion that reinforced concrete structures when subjected to a severe conflagration will be in danger of being so greatly damaged that it will be necessary to rebuild them as a whole or at the least in part. On the other hand, the high non-conductivity of concrete and its incombustibility make it an excellent fireproofing material, which, under the action of an ordinary fire, will remain undamaged and add no fuel to the flames.

Effect of Flue Gases and Moisture on Concrete.—Reinforced concrete has been extensively used for the construction of chimneys in the past few years, but very few data are available as to the action of flue gases upon concrete. Mr. Francis T. Haward, of Silberhütte, Anhalt, Germany, gives the following information in regard to experience at the plant of the Anhaltische Bleiund-Silber-werke, in a paper presented at the Lake Superior meeting of the American Institute of Mining Engineers, in February, 1905:

The flues and smaller stacks at the works were constructed of concrete consisting generally of 1 part of cement and 7 parts of sand and jigtailings (stone culled from the ore), but when reinforcement was used a 1 to 4 mixture was employed.

A continued temperature above 212° F. caused the concrete to crack and ultimately fail. Neutral furnace gases at 250° F. passing through an independent concrete flue and stack, caused so much damage by cracking that after two years of use the stack constructed of concrete pipe 4 ins. thick required thorough repairing and auxiliary ties for every foot of height.

The side of the main flue, made of blocks of 6 in. hollow wall section, about 40 ins. by 20 ins. in area, were covered with 2-in. or 1-in. slabs of reinforced concrete. In cases where the flue was protected by a wooden or tiled roof, and inside by an acid-proof paint, consisting of waterglass and asbestos, the concrete was not appreciably affected. In another case, where the protective coat, both inside and outside, was of asphalt only, the concrete was badly corroded and cracked at the end of three years. In a third case, in which the concrete was unprotected from both atmospheric influences on the outside and furnace-gases on the inside, the flue was quite destroyed at the end of three years. That portion of the protected concrete flue near the main stack which came in contact only with dry, cold gases was not affected at all.

Gases alone, such as sulphur dioxide, sulphur trioxide and others, do not affect concrete; neither is the usual quantity of moisture in furnace gases sufficient to damage concrete: but should moisture penetrate from the outside of the flue, and, meeting gaseous SO_2 or SO_3, form hydrous acids, then the concrete will be injured. Moisture alone does not injure concrete, but moisture mixed with flue gases will cause great damage. It is also stated that soluble salts, noticeably zinc sulphate, when let fall upon concrete, will penetrate it and, by crystallizing, cause the concrete to crack and shell off.

If a concrete chimney is lined to such a height that the flue gases acting directly upon the concrete do not exceed about 200° F., no danger need be apprehended. The many chimneys of reinforced concrete now in use would lead one to infer that in a majority of cases this material is satisfactory.

CHAPTER X.

THE GENERAL ELASTIC PROPERTIES OF CONCRETE.

The strength of concrete varies greatly. This is due to a number of causes, among which the following are the most important: (1) the quality and amount of cement used; (2) the kind, size and strength of the aggregate; (3) the thoroughness with which the ingredients are balanced (the most dense concrete giving the greatest strength); (4) the method of mixing and the thoroughness with which it is done, and (5) its age. To a certain extent the strength of concrete varies with the amount of water used in mixing, the amount of tamping done in depositing, and the hygrometric state of the atmosphere during setting.

The above five items will only be considered as regards the compressive strength and elasticity of concrete, as concrete is seldom or never used in tension.

Tensile Strength.—The ultimate strength of concrete in tension seems to vary in some manner with the richness of the mixture and the age of the specimen, but thus far experimenters have not been able to determine the relations which these quantities bear to each other, nor have they been able to determine any definite elastic limit at a point less than the ultimate strength. Prof. Talbot gives the values shown in Table XXXV. for the ultimate tensile strength of a 1 : 3 : 6 concrete.*

TABLE XXXV.—SHOWING STRENGTH OF CONCRETE IN TENSION.

Test No.	Age. Days.	Mixture.	Maximum Load. Lbs. per sq. in.	Remarks.
7	50	1 :3 :6	178	Bending at start.
3	60	1 :3 :6	160	Bending throughout.
13	84	1 :3 :6	170	Bending throughout.
12	87	1 :3 :0	278	Little bending.

The amount of deformation in these specimens varied from 0.00005 to 0.00006 of the length when the piece broke. Prof. Hatt states that his recent tests show the tensile strength of 1 : 2 : 4 concrete at 28 days to be about 300 lbs. per sq. in. He also gives the strength of 1 : 2 : 5 concretes, which are tabulated

*Bulletin of the University of Illinois for September, 1904.

in Table XLIX., see page 204. Herr Saunders, in some of his tests, obtained tensile strength ranging from 310 to 450 lbs. per sq. in. for specimens one month old, and from 400 to 510 lbs. for concrete three months old, the proportions of the material being 1 : 2 : 2 gravel concrete.

Tests were made by Prof. Ira H. Woolson, at Columbia University, to determine the relative tensile strengths of large test pieces made with full sized stone, and small sized specimens made with sand and with crushed limestone. These tests gave the following results for small specimens of standard briquette size:

	7 days. lbs.	28 days. lbs.
1 Cement, 3 Sand	175	249
1 Cement, 3 Crushed Limestone	345	461

The sand used was a fair grade of moderately sharp sand, and contained less than 1 per cent. of loam. It all passed freely through a 1/8-in. screen. More than 75 per cent. of it would pass a 20-mesh sieve. The limestone was a Hudson River bluestone and all passed a 1/8-in. screen.

The large specimens were 6 × 6 ins. in cross-section and 3 ft. 6 in. long.

During testing little or no bending took place, nearly all the specimens breaking within the middle third. The following results were obtained from one series of tests on a 1 : 2 : 4 crushed limestone mixture.

Days in Mould.	Days in Water.	Days in Air.	Strength. Lbs. per sq. in.
1	6	21	267
1	6	21	282
1	6	17	238
3	6	24	130
2	7	26	250
2	7	26	201
Average strength			228

The broken stone for the large specimens all passed a 1¼-in. screen and remained on a 3/8-in. screen. The concrete was carefully tamped into the moulds in a moderately wet condition. These experiments would seem to indicate that the tensile strength of a 1 : 2 : 4 limestone concrete, made with full-sized stone, is slightly less than that of briquettes made from a 1 : 3 sand mixture, and about one-half the strength of briquettes made from a 1:3 crushed limestone mixture. Freshly broken stone on the fractured face would seem to indicate that the tensile strength of the concrete was approximately that of the stone. More data on this subject are greatly to be desired.

In general, for a 1 : 2 : 4 concrete the ultimate tensile strength for one month may be taken at about 300 lbs., and for three months from 400 to 500 lbs.; for mixtures ranging from 1 : 2 : 5 to 1 : 3 : 6, for one month, 150 to 200 lbs., and for three months, from 200 to 350 lbs. per sq. in.

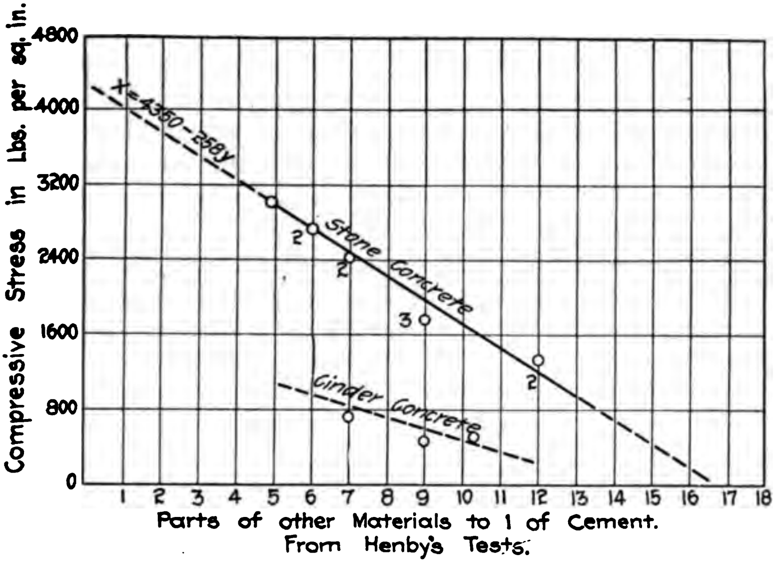


Fig. 64.—Diagram Showing Effect of Amount of Cement on Compressive Strength.

Working Stresses.—When concrete is to be used in tension working stresses not greater than from one-fifth to one-sixth of the above values should be employed, giving a working stress of from 30 to 100 lbs. per sq. in., depending upon the age and richness of the mixture.

Effect of Amount of Cement on Compressive Strength.—The ultimate compressive strength of concrete decreases uniformly as the proportion of cement in the mixture decreases. Fig. 64 is a diagram in which the compressive resistances of Henby's tests* have been plotted and the results averaged by means of a

*Proceedings of Association of Engineering Societies, September, 1900.

straight line. This diagram illustrates the relation between the strength and parts of materials to one part of cement, under certain conditions.

This is also shown by Rafter's tests, described on page 193, and shown in Table XLV. The variations in strength of dry and plastic mixtures for different proportions are as follows:

	Proportions.	Comp. Strength.
Dry mixture	1 :1 :5	4,659
	1 :2 :6	3,686
	1 :2 :7	2,800
Plastic mixture	1 :1 :4	4,462
	1 :2 :6	3,400
	1 :2 :7	3,132

Too much credence should not be given these tests in this connection, as different brands of cement were used.

Kimball's and the Watertown Arsenal tests of 1901 and 1904, shown in Tables XXXVI. to XL. and in Table XLIV., should also be consulted in this connection.

Effect on cinder concrete is also shown in Table XL.

Effect of Variation in Size of Stone.—The variation in ultimate compressive strength due to varying sizes of stone and gravel is shown in Table XXXVI., taken from the Watertown Arsenal Report for 1901. This table shows the ultimate crushing strength of 12-in. cubes. The cement used was Alpha Portland; the other ingredients are shown in the table.

TABLE XXXVI.—SHOWING CRUSHING STRENGTH OF 12-IN. CUBES.

(From Watertown Arsenal Tests, 1901.)

Composition.			Age.		Ult. Resist. in
Cement.	Sand.	Broken Stone.	Years.	Months.	Lbs. per sq. in.
1	2	4 ¾-in. Trap.	1	..	3,187
1	3	6 " "	1	..	2,070
1	4	8 " "	1	..	1,499
1	5	10 " "	1	..	949
1	6	12 " "	1	..	791
1	2	4 1½ and 2½-in. Trap	2	..	2,789
1	2	4 1 and 2½-in. Trap	2	..	2,549
1	2	4 2½-in. Trap	2	..	2,466
1	2	7 " "	1	2	2,406
1	2	4 1½ to 3-in. Pebbles.	1	2	3,589

Table XXXVII., taken from the same report, shows the ultimate crushing strength of 6 × 6 × 36-in. prisms with an average age of 33 days:

TABLE XXXVII.—CRUSHING STRENGTH OF 6 × 6 × 36-IN. PRISMS.

(From Watertown Arsenal Tests, 1901.)

Cement.	Sand.	Stone.	Number of Specimens Tested.	Ult. Crushing Strength in Lbs. per. sq. in.
1	2½	4 ½-in. to 2-in. diam. Pebbles.	8	2,326
1	2½	4 ¼-in. to 2½-in. diam. Gravel.	6	3,363
1	2½	4 1-in. to 2½-in. diam. Hard Trap Rock.	6	3,886

Table XXXVIII. shows the effect of the variation in size of stone and gravel for a 1 : 1 : 3 mixture, the cement used being Alpha brand. Later tests, shown in Tables XXXIX. and XL., give the variations in strength from extremely rich to as lean mixtures as are used in concrete work. These latter tables are taken from a table of tests made in the Watertown Arsenal in 1904. These tests show the effect of varying the sizes of the stone and gravel.

TABLE XXXVIII.—EFFECT OF SIZE OF STONE ON STRENGTH OF CONCRETE.

(From Watertown Arsenal Tests, 1898.)

12-inch Cubes 1 :1 :3 Alpha Cement.

Size and kind of Aggregate.	Compressive Strength in lbs. per sq. in. at age in days. 7-8	19-23	29-34	61-76	Coefficient of Elasticity in lbs. per sq. inch. Between loads of 100 and 600 lbs. per sq. in. About 1 mo.	About 2 mos.
½-in. trap rock	1,391	2,220	2,800	5,021	3,571,000	4,167,000
¾-in. " "	1,900	2,769	3,200	....	8,333,000	....
1 -in. " "	3,390	4,254	4,917	5,272*	6,250,000	8,333,000
1½-in. " "	3,189	4,006	4,562	2,583	....	....
2½-in. " "	2,400	4,143	4,140	4,523	5,000,000	12,500,000
½-in. " " 1 part; 2½-in. " " 2 parts	2,800	3,786	4,340	4,544*	8,333,000	6,250,000
½-in. " " 1 part; 1 -in. " " 1 part; 2½-in. " " 1 part	2,800	4,156	4,800	5,542*	8,333,000	8,333,000
Mean strength trap rock	2,553	3,619	4,110	4,581		
Gravel, ⅜-in.	1,298	2,600	2,992	3,870	4,167,000	3,125,000
" 1½-in.	2,276	3,186	3,817	4,018	4,167,000	2,778,000
" ⅜-in., 1 part; " 1½-in., 2 parts	1,994	3,023	3,800	3,490	4,167,000	5,000,000
" ⅛-in., 1 part; " ⅜-in., 1 part; " 1½-in., 1 part	1,486	2,676	3,000	3,800	3,125,000	3,125,000
Mean strength gravel	1,764	2,871	3,402	3,794		

*Not fractured.

TABLE XXXIX.—SHOWING EFFECT OF SIZE OF STONE ON STRENGTH OF CONCRETE.

(From Watertown Arsenal Tests, 1904.)

Average age, 173 days.

Mixture			Compressive Strength.			
Cement.	Sand.	Gravel.	½-in. Trap.	2½-in. Trap.	¼-in. Gravel.	⅝-in. Gravel.
1	..	1	6,400	....	....	....
1	..	1	....	....	4,800	....
1	..	1	....	....	....	4,360
1	..	1	....	4,200	....	....
1	1	2	4,180	2,200	....	2,600
1	2	3	3,700	1,680	....	2,060
1	2	4	1,480	1,210	....	1,700
1	3	6	1,410	790	....	580

Table XXXIX. also shows comparative strength of stone and gravel concrete.

TABLE XL.—SHOWING STRENGTH OF CINDER CONCRETE.

(From Watertown Arsenal Tests, 1904.)

Mixture			Age	Strength	Percentage
Cement.	Sand.	Cinders.	in Days.	in Lbs.	of Water.
1	..	1	174	2,930	35.
1	1	2	171	2,200	29.2
1	2	3	171	1,400	43.8
1	2	4	171	800	57.1
1	3	6	171	740	80.0

It should be noted that a very high percentage of water is needed in mixing cinder concrete.

Thus we see, if the ratio of cement to the aggregate remains constant, a variation in the size of stone used may give a different balancing of the mixture and directly affect the ultimate crushing strength.

Effect of Mixing.—The effect on the strength of concrete due to the thoroughness in mixing may to some extent be understood by referring to page 70, where hand vs. machine mixing is discussed.

Effect of Amount of Water.—In the past, considerable difference of opinion has been held by engineers regarding the amount of water that should be used in mixing mortars and concretes. Some years ago it was considered that concrete should be mixed dry and thoroughly tamped to secure the best results. At the present time, however, it is almost universally conceded that wet concretes possess practically as great if not greater strength than dry concretes. The saving in labor for tamping when wet mixtures are used is a very important item in the cost of construction. This alone, even if considerable less strength results, would compel the use of wet concretes. The greater ease with which wet mixtures are placed in reinforced concrete work, and the greater assurance that the metal will be thoroughly surrounded in all cases, also make the use of wet mixtures almost imperative.

Tests on mortars almost invariably show a reduction in strength with the increase in the percentage of water used. T. S. Clark gives the following evidence in Table XLI.:

TABLE XLI.—EFFECT OF PERCENTAGE OF WATER ON STRENGTH OF MORTAR BRIQUETTES.

Age in days.	Percentage of water.	Average tensile strength lbs. per sq. in.	Age in days.	Percentage of water.	Average tensile strength lbs. per sq. in.
7	18	722	28	18	684
7	20	680	28	20	762
7	22	638	28	22	809

The effect of different percentages of water on neat cement and mortar briquettes is also shown in Table XLII., taken from the report of Chief Engineer U. S. Army, 1898:

TABLE XLII.—EFFECT OF VARYING PROPORTIONS OF WATER.

(From Report Chief Engr. U. S. Army, 1898.)

Mortar, Proportions.	Water, Per cent.	Strength 7 Days.	28 Days.	6 Months.
Neat	18.66	715	728	...
Neat	20.83	602	681	...
1:1	13.31	492	639	...
1:1	15.00	384	624	...
1:2	12.00	330	525	...
1:2	12.37	288	470	...
1:3	11.00	229	385	...
1:3	12.50	182	312	...
1:3	7.36	250	308	371
1:3	8.25	271	321	437
1:3	12.50	168	242	343

The results of a series of tests made by J. W. Sussex, and published in the "Technograph" of the University of Illinois, 1903, are shown in Table XLIII. These tests were made on a concrete mixture composed of 1 part Portland cement, 3 parts sand, 6 parts crushed limestone. Forty-five (45) 6-in. cubes, mixed with three different percentages of water, were broken at the end of 7 days, 1 and 3 months. Two different degrees of tamping were also employed. Each result shown is an average of three tests. As will be seen, the wet concretes at the end of 3 months furnish the greatest ultimate resistance, although at the earlier periods the medium concretes gave higher results. The effect of tamping does not materially modify the relative strength as affected by the degree of wetness. **Rafter's tests should also be** consulted in this connection.

TABLE XLIII.—EFFECT OF VARYING PERCENTAGE OF WATER ON 1 : 3 : 6 CONCRETE CUBES.

(Tests made by J. W. Sussex.)

	Crushing Strength in lbs. per sq. in.				
	Dry, 6% water.		Medium, 7.8% water.		Wet, 9.4% water.
Age.	Lightly tamped.	Heavily tamped.	Lightly tamped.	Heavily tamped.	
7 days	1,200	1,340	2,280	1,330	1,040
1 month	1,750	1,960	2,290	2,560	2,230
3 months	2,500	2,600	2,150	2,590	3,040

Effect of Age.—The amount of increase in strength of concrete from 7 days to 6 months is shown in Table XLIV., which is taken from tests made by George A. Kimball on 12-in. cubes at Watertown Arsenal in 1899.

TABLE XLIV.—SHOWING INCREASE OF STRENGTH WITH AGE.

(From Kimball's Tests.)

	Crushing Strength in lbs. per sq. in.			
Mixture.	7 Days.	1 Month.	3 Months.	6 Months.
1 :1 :3	1,600	2,750	3,360	4,300
1 :2 :4	1,525	2,460	2,944	3,900
1 :2½ :5	1,300	2,225	2,670	3,400
1 :3 :6	1,230	2,060	2,440	3,100
1 :3½ :7	1,100	1,875	2,210	2,800
1 :4 :8	1,000	1,700	1,980	2,500
1 :5 :10	800	1,350	1,520	1,900
1 :6 :12	600	1,000	1,060	1,300

Table XLIII. should also be consulted in this connection.

The average of Kimball's tests, which were carefully made, would indicate a compressive strength for 1 : 2 : 4 concrete of 2,400 lbs. at 1 month, 3,000 lbs. at 3 months and nearly 4,000 lbs. at 6 months; and for 1 : 3 : 6 concrete, 2,000 lbs. at 1 month, 2,400 lbs. at 3 months and 3,000 lbs. at 6 months.

From Kimball's tests Mr. Thacher has deduced formulas for the ultimate strength of concretes. These formulas give results which agree very well with the experiments, and may be used for obtaining the strength of concretes carefully made from good materials.

Thacher's Formulas.—The ultimate strength in pounds per square inch of concrete:

$$\text{7 days old} = 1{,}800 - 200\left(\frac{\text{Volume of Sand}}{\text{Volume of Cement}}\right)$$

$$\text{1 month old} = 3{,}100 - 350\left(\quad \text{''} \quad\right)$$

$$\text{3 months old} = 3{,}820 - 460\left(\quad \text{''} \quad\right)$$

$$\text{6 '' ''} = 4{,}900 - 600\left(\quad \text{''} \quad\right)$$

In addition to the tests already given, a series of tests made by Prof. Woolson, at Columbia University, deserves mention. A 1:2:4 concrete made from crushed limestone and tested at 30 days, gave an average of 2,450 lbs. per sq. in. These tests agree quite well with those already given. Details of a number of these tests are given on page 207.

It should be noticed that according to Mr. Thacher's formulas the strength of concrete depends upon the ratio of the volume of sand to the volume of cement. A number of authorities hold that this ratio, rather than the ratio of aggregate to cement, correctly determines the variation in strength of concretes. If the aggregate be of good, clean material, so graded as to sizes that there will be a minimum of voids, the strength can be safely said to depend upon the richness of the mortar used, provided an excess of mortar over the remaining voids be used, always bearing in mind that it is practically impossible to fill all voids. The amount of mortar necessary to fill the voids, and therefore the ratio of cement to total aggregates, thus finally determines the strength. Finally, we must conclude that with a well balanced mixture, *i. e.*, where the voids are a minimum, the strength will depend upon the richness of the cementing material or the ratio of sand to cement, but under ordinary conditions it is safest to consider that the strength depends upon the ratio of aggregates to the cement.

Rafter's Tests.—Probably the most exhaustive tests made on concrete in this country were conducted by George W. Rafter and recorded in the Report of the State Engineer of New York for 1897. Compressive tests were made on 544 12-in. cubes whose age at the time of testing averaged about 600 days. The concrete was prepared in three different ways: (1) in dry blocks in which the mortar was only a little more moist than damp earth, (2) in plastic blocks, the mortar of which was of the consistency of that used by masons, and (3) in blocks having an excess of water, so that the mortar quaked like liver when moderately rammed. Specimens of each batch were prepared and stored differently. One block was placed in water for about 4 months and then buried in sand until shipped to Watertown Arsenal for testing. The second stood in a cool cellar until shipment. The third block was exposed to the weather, and the fourth block was covered with burlap and wet with water several times a day for about three months, and then exposed to the weather until the day of shipment. Portland cement was used. The aggregate was a hard, broken sandstone.

A careful examination of the results of the tests shows a great uniformity between the blocks of different groups, the method of storage not seeming to affect the strength of the concrete. The dry mixtures show a slight increase in strength over the wet, but it is not of much importance, and in practice the additional cost of tamping dry concrete would more than offset the gain of strength secured thereby. Another deduction from these tests is that the strength of the concrete increases with the richness of the mortar. Some of these tests are shown in Table XLV.

TABLE XLV.—RAFTER'S TESTS OF 12-IN. CONCRETE CUBES.

Brand of Cement.	Approximate Proportions.	Consistency.	Ultimate Strength lbs. per. sq. in.	Modulus of Elasticity 100-600 lbs.	100-1,000 lbs.	1,000-2,000 lbs.
Genesee	1:1:5	Dry	4,659	3,571,000	2,812,000	1,667,000
Wayland	1:1:4	Plastic	4,462	4,167,000	2,500,000	1,219,000
Wayland	1:2:6	Dry	3,686	2,083,000	1,875,000	1,111,000
Ironclad	1:2:6	Dry	4,000	3,125,000	3,000,000	2,000,000
Champion	1:2:6	Dry	2,598	3,125,000	2,812,000	1,398,000
Champion	1:2:6	Plastic	2,400	2,500,000	2,143,000	1,163,000
Ironclad	1:2:6	Plastic	3,600	3,571,000	3,000,000	2,000,000
Genesee	1:2:6	Plastic	3,400	3,125,000	2,812,000	1,667,000
Empire	1:2:6	Excess	2,950	2,500,000	2,045,000	1,163,000
Ironclad	1:2:6	Excess	3,436	3,125,000	2,812,000	1,471,000
Champion	1:2:7	Dry	2,800	3,125,000	2,812,000	1,724,000
Ironclad	1:2:7	Dry	3,283	2,500,000	2,368,000	1,282,000
Wayland	1:2:7	Plastic	3,132	2,778,000	2,250,000	1,429,000
Empire	1:2:7	Excess	3,400	3,571,000	2,647,000	1,250,000

Elastic Limit.—Concrete in compression has a fairly well defined point of elastic limit. This has been given as a little more than one-half of the ultimate strength. Henby, in the tests mentioned above, places the elastic limit at two-thirds of the ultimate strength. The limit is probably somewhere between these points and the ultimate strength.

Working Stresses.—The proper working stress to be used in reinforced concrete work depends upon the character of the structure; the nature of the stress, *i. e.*, whether it is a direct or bending stress; whether the load is a dead or a live load; whether it acts directly or through the medium of some inert material, and lastly upon the richness and age of the concrete. No fixed rule can be given for selecting the unit working stress, as each structure is a problem requiring separate treatment by the engineer. The following values for factors of safety will give conservative working stresses and lead to safe designs:

A factor of safety of 5 or 6 should usually be chosen for concrete having an age of 3 months, although under exceptional conditions, with uniform loads, and when there is no vibration or impact, as low a factor as 4 is sometimes used. These factors are to be used with the proper ultimate strength for the given mixtures, as determined by Mr. Thacher's formula, or chosen from some one of the tables cited. It is probable that the full load will be brought upon the given structure at the end of 3 months, and it is often desirable if not necessary that it shall be supported at the end of 1 month or 6 weeks. Any additional strength after 3 months may be neglected, as the critical period of straining will then be past.

If we assume a 1:2:4 concrete having a strength of 3,000 lbs. at 3 months, and 2,400 lbs. at 1 month, and use a factor of 6 at 3 months, we will have a working stress of 500 lbs. at 3 months and a factor of safety of 4.8 at the end of 1 month. In like manner, for a factor of 5 at 3 months, we have at that age a working stress of 600 lbs. and a factor of safety of 4 at 1 month. If the factor at 3 months be 4, the working stress will be 750 lbs., and the factor of safety at 1 month will be 3.2. In like manner, a 1:3:6 concrete having an ultimate strength of, say, 2,400 lbs. and 2,000 lbs. at 3 months and 1 month, respectively, will give for a factor of safety of 6 a working stress of 400 lbs. at 3 months and a factor of 5 at 1 month. For a factor of safety of 5, a stress of 480 lbs. at 3 months and a factor of 4.2 at 1 month, and for a factor of 4, a stress of 600 lbs. at 3 months and 3.3 for a factor of safety at 1 month. Under normal conditions these factors will give ample strength.

The manner in which the concrete is subjected to compression modifies somewhat the working values which should be used. The above values may be employed for compression under bending. For concrete used in direct compression, as in columns, values of about 80 per cent. of the above should be employed.

Cinder Concrete.—For roof slabs and floors of buildings, and in other situations where light weight is desired, cinder concrete is sometimes used. The strength of cinder concrete is considerably less than that of stone concrete. Tables XL. and XLVI. give the results of a number of tests made at Watertown Arsenal for the Eastern Expanded Metal Co., of Boston, and show the average of a number of tests. Steam cinders were used, practically as they came from the furnace, the large clinkers being broken up.

Table XLVII. from Watertown Arsenal tests of 1903 and 1904 gives more recent tests on cinder concrete.

These values appear to be somewhat high.

Table LVI. shows the compressive strength of cinder concretes, age 30 and 60 days, of three different proportions, together with their coefficients of elasticity. These are taken from Henby's tests, and each value represents the average of a number of tests. Table LV. also gives compression strengths for cinder concrete of different proportions. From the above tests we may infer that the strength of good cinder concrete is about 0.4 that of stone concrete. The strength of cinder concrete is, however, much more variable than that of stone concrete. Thus, for a 1 : 1 : 3 concrete, a mixture frequently used for reinforced concrete floors, the strength, as shown by Tables

TABLE XLVI.—COMPRESSIVE STRENGTH OF 12-IN. CUBES OF CINDER CONCRETE.

(Watertown Arsenal Tests.)

Brand of cement.	Mixture.	Average compressive strength, lbs. per sq. in. Age 1 month.	Age 3 months.
Germania Portland.........	1:1:3	1,466	2,001
" "	1:2:3	1,098	1,634
" "	1:2:4	904	1,325
" "	1:2:5	769	1,084
" "	1:3:6	529	788
Alpha Portland............	1:1:3	2,329	2,834
" "	1:2:5	940	1,600
Atlas Portland.............	1:1:3	1,601	2,414
" "	1:2:5	696	1,223

TABLE XLVII.—COMPRESSIVE TESTS ON CINDER CONCRETE CUBES.

(Watertown Arsenal Reports of 1903 and 1904.)

Proportions			Age		
Cement.	Sand.	Cinder.	38 days.	224 days.	1 year 3½ months.
1	2	4	1,950	....	2,440
1	2	4	2,050	....	2,490
1	2	4	....	2,600	2,610
1	2	4	....	2,500	2,410
1	2½	5	1,400	....	1,950
1	2½	5	1,400	....	1,630
1	2½	5	....	1,980	1,480
1	2½	5	....	2,020	1,740
			34 days.	220 days.	
1	3	6	1,200	1,730	1,400
1	3	6	1,330	1,560	1,380
1	3	6	....	....	1,290
1	3	6	....	....	1,380

XLVI. and LV., varies from 2,001 to 2,834 lbs., while for a 1:2:5 concrete, also a common mixture, it varies from 1,200 to 1,715 lbs. The uncertain strength of cinder concrete, and the slovenly manner in which it is usually prepared, make a high factor of safety imperative in determining working values for this material. Factors as high as from 6 to 10 should be used. These will give working stresses of from 100 to 400 lbs. per sq. in., depending upon the age, nature and richness of the mixture.

Transverse Strength of Concrete.—The tensile strength of concrete at the place of greatest strain, that is, at the fibre most remote from the neutral axis, limits the strength of unreinforced concrete beams. The value of this transverse strength is of little importance, because, on account of the brittleness of concrete in tension, its liability to crack from shrinkage or the shock of some of the applied loads, it is unsafe to depend upon the tensile strength of concrete for the construction of slabs, beams or girders. For the same reason, it is now common practice to disregard in the design of reinforced concrete beams the tensile strength of the concrete. Under certain conditions, however, it may be necessary to take the strength of concrete in tension into consideration, as in the case of a foundation to be placed under water, when it is necessary to absolutely insure the protection of the metal from any possible contact with water. Under such conditions, stress in the steel and the resulting deformation should be kept so low that no perceptible cracks will result in the concrete, always retaining the proper ratio between the modulae of elasticity of the two materials.

Sabin states that the ratio of the transverse to the tensile strength of concrete varies from 1.25 to 1.90 for Portland, and from 0.95 to 2.19 for natural cement concrete.

Shearing Strength of Concrete.—The subject of shearing strength of concrete needs careful experimental study. The following facts are gleaned from the little knowledge on the subject available:

M. Mesager, Director of the School of Bridges and Roads, Paris, gives as a result of his experiments the shearing strength of concrete at from 1.2 to 1.3 times its tensile strength. This agrees very well with the researches of Herr Bauschinger, who states that the shearing strength of concrete 4 months old is 1.25, and at 2 years 1.5 times its tensile strength. M. Feret, Director of the Laboratory of Bridges and Roads at Boulogne, has studied the shearing strength of concrete, and concludes that the ultimate shearing strength is proportional to the compressive strength, and gives its value at from 16 to 20 per cent. of its compressive strength. His results do not differ materially from those already given. All other results hitherto available agree well with those just stated.

Details of the method of conducting the above tests are not available. It is probable, however, that the specimens were subjected to more or less bending, hence the low values obtained are more nearly tension values, as what were taken as shearing failures were really diagonal tension failures. Prof. Arthur N. Talbot states, in Bulletin No. 4 of the University of Illinois, that from tests made at the University of Illinois and elsewhere it is probable that the shearing strength of concrete is from 50 to 75 per cent. of its compressive strength.

Massachusetts Institute of Technology Tests.—Series of tests have been conducted at the Massachusetts Institute of Technology. The 1904 and 1905 tests were conducted by Prof. Charles M. Spofford, and the 1906 tests by Prof. F. P. McKibben, the tests being under the general direction of Prof. Swain, Professor of Civil Engineering. The author is indebted to Professors Swain and Spofford for details of these tests.

The test specimens were 5 ins. in diameter by 15½ ins. in length, and in testing were firmly held in cylindrical bearings 5½ ins. apart. The load was applied from above through a half cylinder bearing 5$^7/_{16}$ ins. in length, so as to eliminate bending as far as possible. Tests were made on specimens which had set in air and on others which had set in water. The latter specimens set in air for 24 hours before being placed in water, while the air set specimens were sprinkled for six days after being removed from the moulds. The specimens were made from a cement composed of a mixture of several standard brands of Portland cement, a sand composed of equal volumes of Plum Island and Ipswich sands, and of a stone composed of a mixture of one volume of ½-in. and two volumes of 1½-in. Waltham trap rock. Roxbury pudding stone instead of Waltham trap rock was used for the 1904 specimens. The mixing was done in small batches and with great care. Crushing tests were made on 6-in. cubes. The values obtained for the 1904 and 1905 tests are as follows:

	1904. Age. days.	1904. Compressive strength, lbs. per sq. in.	1905. Age. days.	1905. Compressive strength, lbs. per sq. in.
1 : 2 : 4	33	2,457	30	2,070
1 : 3 : 5	24	1,225	30	1,355
1 : 3 : 6	27	1,104	27	1,275

A summary of the results of the tests for shearing strength is shown in Table XLVIII., together with the ratio of air set to water set specimens. As will be noted, there is considerable variation in the shearing strength of the weaker specimens. By comparing the shearing values here given with the compressive values given in the preceding table, it would appear that the shearing strengths are slightly greater than 50 per cent. of the compressive strength.

TABLE XLVIII.—SHEARING STRENGTH OF CONCRETE.

Mixture.	Manner of set.	Results, 1904.	Results, 1905.	Results, 1906.	Av.	Air set / Water set =
1 : 2 : 4	Water.	1,192	1,649	1,397	1,427	0.84
1 : 2 : 4	Air.	973	1,314	1,200	1,192	
1 : 2½ : 5	Water.	....	....	879	....	0.89
1 : 2½ : 5	Air.	....	....	780	....	
1 : 3 : 5	Water.	579	1,121	....	850	1.04
1 : 3 : 5	Air.	541	1,236	....	889	
1 : 3 : 6	Water.	599	1,123	701	808	1.00
1 : 3 : 6	Air.	629	1,185	615	808	

Safe Working Values for Shearing.—Christophe, in Bêton Armé, gives as safe working values from 14 to 35 lbs. per sq. in. These are somewhat lower values than are used in this country, values of from 40 to 75 lbs. being usually allowed. Of course, the richness and age of the concrete must be taken into account in choosing the working strength for shear. When shearing stresses greater than the above assumed values are met with in reinforced concrete work, special provisions, as by the use of stirrups, must be had recourse to.

Modulus of Elasticity.—The coefficient or modulus of elasticity of a material for tension, compression or shear is the ratio of the unit stress to the unit deformation, provided the elastic limit of the material be not exceeded. This coefficient is usually denoted by

$$E = \frac{f}{\epsilon}$$

in which expression f designates the unit stress and ε the unit deformation. The stress-strain relation throughout the entire range of stress may be clearly shown by means of a diagram in which the stresses in pounds per square inch are plotted as ordinates, and the deformation per inch as abscissas. This will be understood by referring to Fig. 65. Thus the curve B will be found to pass through the line representing a compressive stress of 1,200 lbs. per sq. in. near its intersection with the deformation line marked 5, which represents a decrease in length per inch of specimen of 0.0005 ins.

The coefficient of elasticity of concrete is not a practically con-

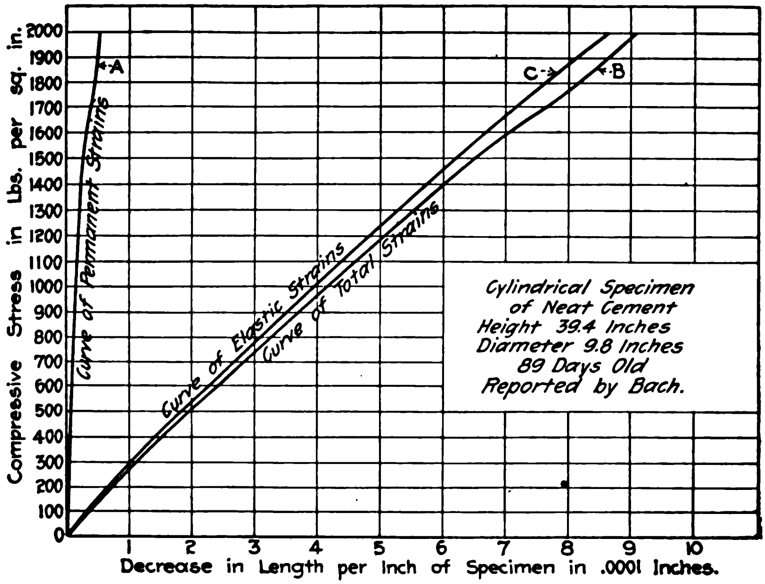


Fig. 65.—Elastic Deformation Curve.

stant quantity, like that for iron or steel, but for a given concrete has a value which varies with the load. As in some other materials, it has been found that concrete takes permanent sets under very light loads. This involves a slight modification in calculating the true value for the coefficient of elasticity, as these sets must be deducted from the total deformation under gradually increasing loads to obtain the true elastic deformation. To obtain accurate results, the load must be applied and removed until no further permanent set can be observed when a loading is applied and removed. The total permanent set noted is then deducted from the total deformation observed for the maximum loading used in the given experiment to obtain the true elastic shortening. Figure 65 shows this method of determining the elastic deformation.

Three curves are shown, curve marked A showing the permanent strains or deformation, curve B the total strains, and curve C the true curve of elastic strains. The latter curve is obtained by subtracting the amount of permanent set shown in curve A from the total strains as shown in curve B.

Many engineers do not take the permanent set into account in computing the coefficient of elasticity of concrete. Curve B will represent the coefficient when thus considered, and may be represented analytically by the expression

$$E = \frac{f}{e}$$

Prof. Bach, of Stuttgart, who has made the most elaborate researches thus far undertaken to determine the coefficient of elasticity of concrete, thinks the true curve of elastic strain may be expressed by an algebraic equation having the form of

$$E = \frac{f^n}{e}$$

n being a numerical coefficient usually considered as having a value of 1 up to the elastic limit, as the curve is represented by a straight line up to that point.

One of the usual methods of calculating E is by determining what might be called the instantaneous value of the coefficient of elasticity. This is done by finding the elastic strain occurring between any two applied loads and assuming that the curve is a straight line between two such points of loading. There results no appreciable error if the points chosen are sufficiently near together.

The following computation shows a usual method of computing the modulus of elasticity:

Let E = the modulus of elasticity.
P = total load in pounds.

A = area of cross-section of test piece.
l = length of specimen in inches throughout which the deformation is uniform.
λ = deformation in inches in the length, l.

$$\epsilon = \text{unit deformation} = \frac{\lambda}{l}.$$

$$f = \text{unit stress} = \frac{P}{A}.$$

$$\text{But } E = \frac{\text{unit stress}}{\text{unit deformation}} = \frac{f}{\epsilon} = \frac{P}{A} \div \frac{\lambda}{l} = \frac{Pl}{A\lambda}.$$

Let us assume a given test from which we have the following data:

Concrete 1:2:4, age 3 months.
Sectional area of test piece = 150 square inches.
Gage length of test piece = 5 inches.
Initial load, 15,000 lbs., gives 100 lbs. stress with 0 compression and 0 set.
Applied load of 90,000 lbs. gives 600 lbs. per square inch stress with a total compression of .00093 inches and a set of .00030 inches.
Then we have

$$l = 5 \text{ and } \frac{P}{A} = 600 - 100 = 500.$$

$$\lambda = .00093 - .00030 = .00063.$$

and

$$E = \frac{P}{A} \times \frac{l}{\lambda} = \frac{500 \times 5}{.00063} = 3{,}968{,}000 \text{ lbs. per square inch.}$$

In eliminating the inelastic deformations, Prof. Bach removed the loading at least five times, and sometimes more. Such a complete elimination of set is probably not obtained in actual structures. In the "Watertown Arsenal Tests" the usual method is to determine the total deformation and permanent set from no load to the ultimate strength. The increments of load are 100 lbs. per sq. in. up to 1,000 lbs., and from that point 200 lbs. per sq. in. For each recorded load the total compression was read, the load removed and set measured, the load repeated and increased to the next higher one, and so on. It is probable that results determined from these tests are more nearly equivalent to those existing in actual structures than those obtained by Prof. Bach.

Numerous experiments differing greatly among themselves have been made to determine the coefficient of elasticity of concrete, both in tension and compression. Falk's "Cements, Mortars and Concretes" contains a review of many of the later experiments. Still later ones may be found described in recent numbers of the "Engineering News" and "Engineering Record." We will give the results deduced from the later experiments. This will enable the student to obtain briefly some knowledge of the best known facts and principles in regard to concrete. Experimental data of the proper character are not available for a thorough understanding of all the properties of reinforced concrete. The subject has been taken up for exhaustive study by a Special Committee of the American Society of Civil Engineers. Tests will be made along definite lines and conducted in a uniform manner, and undoubtedly when the results are available a much more thorough understanding of the subject will be possible.

Effect of the Density of Concrete and the Amount of Water Used in Mixing, on Coefficient of Elasticity.—Considère found that the amount of water used in mixing the concrete had considerable influence on the coefficient of elasticity, its value diminishing when an excess of water is used. Insufficient tamping also causes a decrease in the value of the coefficient, and in general increasing the density of a concrete tends to increase the value of the coefficient of elasticity.

Coefficient of Elasticity of Concrete Under Tension.—The elastic behavior of concrete under tensile stresses is more variable than that under compression. There seems much difference of opinion among different experimenters in regard to the form of the curve of elastic deformation under tensile stresses. Some hold that the variations of the coefficient of elasticity in tension may be neglected by reason of its comparative smallness, and that there is no point which can be taken as the limiting stress. Others hold that for small stresses the coefficient is practically invariable, but for larger ones the increments of elongation are great, and the shape of the elastic curve becomes very flat. It is reasonable to suppose that if the tensile strength varies greatly the deformations will be variable also.

In his earlier tests Prof. Hatt made deductions which caused him to believe that the coefficient of elasticity in tension varied somewhat from the coefficient of elasticity in compression, but from a careful study of more recent tests he concludes that the two coefficients are practically equal. This agrees very well with other late experiments, and for all practical purposes may be thus considered. Table XLIX. gives the average of 37 compression and 27 tension tests, made by Prof. Hatt.* The broken stone was limestone, being crusher run below 1 in., and the gravel was pit gravel, including sand and pebbles.

TABLE XLIX.—MODULI OF ELASTICITY.

(Prof. Hatt's Tests.)

Kind of Concrete				Age, days.	Compression Modulus	Tension Modulus.	Ultimate Strength	
Cement.	Sand.	Broken Stone	Gravel.				Compression.	Tension.
1	2	5	..	90	4,610,000	5,460,000	2,413	359
1	2	5	..	28	3,350,000	3,800,000	2,290	237
1	..	..	5	90	4,800,000	4,510,000	2,804	290
1	..	..	5	28	4,130,000	4,320,000	2,400	253

Table L. gives the moduli of elasticity for a number of tests made on large concrete specimens in tension by Prof. Woolson, at Columbia University. The specimens were made from full sized stone and were broken at about 30 days. For further particulars in regard to these tests see page 207.

M. De Joly states as a result of experiments made by him that no definite point could be determined for the elastic limit, but that it seemed to be very near the point of rupture for neat cement specimens, and that it never fell below three-fourths of the ultimate resistance for mortars or concretes.

TABLE L.—MODULI OF ELASTICITY (E) FOR TENSION.

(Woolson's Tests.)

1:2:4 concrete. Specimens 6 × 6 ins.

(Elastic Curve = Deformation Curve Minus Set Curve.)

Crushed Limestone and Broken Limestone.

Test No.	E at 28 lbs. per sq. inch.	E at 83 lbs. per sq. inch.	E at 135 lbs. per sq. inch.	Breaking Load. lbs. per sq. in.
3012	5,700,000	4,670,000	5,140,000	186
3013	5,170,000	4,085,000	4,310,000	151
3014	6,410,000	5,130,000	4,670,000	150
3015	5,170,000	3,610,000	4,010,000	158
3064	4,660,000	4,050,000	4,011,000	204
Average..	5,422,000	4,309,000	4,428,200	170

Sand and Broken Limestone.

Test No.	E at 28 lbs. per sq. inch.	E at 83 lbs. per sq. inch.	E at 135 lbs. per sq. inch.	Breaking Load. lbs. per sq. in.
3067	4,790,000	4,790,000	4,790,000	153
3068	4,420,000	3,975,000	3,680,000	176
3069	6,275,000	4,710,000	4,400,000	153
Average..	5,161,700	4,491,700	4,290,000	161

*The Journal of Western Society of Engineers, June, 1904, page 234.

The coefficient appears to increase with the ultimate strength of the material, but no definite relation between them has thus far been determined.

Coefficient of Elasticity in Compression.—The coefficient of elasticity increases with the age of the concrete up to about three months. Beyond this time any additional increase may be neglected. Table LI., from Kimball's tests, made at Watertown Arsenal in 1899, will give an idea of this increase.

TABLE LI.

(Kimball's Tests on 12-inch Cubes.)

Composition			Age	Modulus of elasticity between loads, per square inch of			Compressive strength per sq. in., lbs.
Cement	Sand	Broken Stone		100 and 600 lbs.	100 and 1,000 lbs.	1,000 and 2,000 lbs.	
1	2	4	7 days.	2,593,000	2,054,000	1,351,000	1,730
1	2	4	1 mo.	2,662,000	2,445,000	1,462,000	2,567
1	2	4	3 mos.	3,671,000	3,170,000	2,158,000	2,975
1	2	4	6 mos.	3,646,000	3,567,000	2,582,000	3,989
1	3	6	7 days.	1,869,000	1,530,000	....	1,511
1	3	6	1 mo.	2,438,000	2,135,000	1,219,000	2,260
1	3	6	3 mos.	2,976,000	2,656,000	1,805,000	2,741
1	3	6	6 mos.	3,608,000	3,503,000	1,868,000	3,068
1	6	12	1 mo.	1,376,000	....	....	1,146
1	6	12	3 mos.	1,642,000	1,364,000	....	1,359
1	6	12	6 mos.	1,820,000	1,522,000	....	1,592

Portland cement, bank sand and broken conglomerate stone.

The coefficient of elasticity decreases as the unit stress increases. All of the most reliable tests show this decrease, and it should be taken into consideration when designing concrete structures. The nature and amount of this decrease will be understood from a study of Table XLV., page 194, from the Watertown Arsenal Report of Rafter's Tests, 1898-9. Table LI., from Kimball's Tests, also shows the decrease in strength between the limits of loading given.

Table LII. gives more recent tests, made at Watertown Arsenal in 1904. The decrease in the value of the coefficient of elasticity here shown corroborates that given in other tests.

Prof. Ira H. Woolson made a series of experiments at Columbia University for the Astoria Light, Heat & Power Co., to determine the strength of concrete made of full-sized stone under ordinary working conditions. While making these experiments an effort was made to determine the elastic properties of the concrete, both in tension and compression.

An electric contact extensometer was employed for determin-

TABLE LII.—COMPRESSIVE STRENGTH AND ELASTIC PROPERTIES OF 4 × 4 × 24-IN. CEMENT, MORTAR AND CONCRETE PRISMS.

(Prisms set in water after initial set of from 1 to 6 days in air. Vulcanite brand of cement used. Water used stated in per cent. of cement by weight. Elastic properties observed on a gauge length of 12 ins.)

(Watertown Arsenal Tests, 1904.)

No.	Cement.	Sand.	Stone or Cinder.	Kind of Stone or Cinder.	Water, per cent.	Age, days.	Compressive Strength per sq. in.	Modulus of Elasticity, between loads per sq. in. of— 100 and 500	500 and 1,000	1,000 and 2,000	2,000 and 3,000
1	Neat.	..	..	....	20.0	171	6,940	....	5,000,000	3,750,000	3,870,000
2	Neat.	..	..	....	25.0	171	6,490	....	4,286,000	3,429,000	3,870,000
3	1	1	..	....	30.0	173	6,940	....	4,286,000	3,870,000	3,750,000
4	1	..	1	¼-in. gravel.	25.0	173	4,800	....	3,158,000	3,158,000	2,857,000
5	1	..	1	⅜-in. gravel.	23.8	173	4,360	....	3,158,000	3,000,000	2,857,000
6	1	..	1	½-in. trap rock.	22.7	174	6,400	....	5,000,000	4,800,000	4,138,000
7	1	..	1	2½-in. trap rock.	21.7	176	4,200	....	3,333,000	3,529,000	3,750,000
8	1	..	1	cinders.	35.0	174	2,930	....	3,158,000	2,791,000	....
9	1	2	..	....	38.6	173	3,280	....	3,529,000	2,857,000	2,182,000
10	1	3	..	....	40.0	173	2,910	....	2,727,000	2,308,000	....
11	1	1	2	½-in. trap rock.	40.0	173	4,560	....	3,529,000	3,750,000	3,333,000
12	1	1	2	½-in. trap rock.	29.2	173	4,180	....	5,000,000	5,217,000	3,870,000
13	1	2	3	½-in. trap rock.	45.0	173	3,700	....	4,615,000	3,333,000	2,927,000
14	1	2	4	½-in. trap rock.	42.9	175	1,480	....	2,500,000	....	....
15	1	3	6	½-in. trap rock.	40.0	168	1,410	....	1,667,000	....	....
16	1	1	2	2½-in. trap rock.	29.2	171	2,200	....	4,615,000	3,429,000	....
17	1	2	3	2½-in. trap rock.	37.5	175	1,680	....	3,529,000	....	....
18	1	2	4	2½-in. trap rock.	42.9	175	1,210	....	2,400,000	....	....
19	1	3	6	2½-in. trap rock.	50.0	171	790	2,824,000	....	....	....
20	1	1	2	cinders.	29.2	171	2,200	....	2,400,000	1,644,000	....
21	1	2	3	cinders.	43.8	171	1,400	....	1,429,000	....	....
22	1	2	4	cinders.	57.1	171	800	1,412,000	....	....	....
23	1	3	6	cinders.	80.0	170	740	1,091,000	....	....	....
24	1	1	2	¾-in. gravel.	33.3	170	2,600	....	2,727,000	2,353,000	....
25	1	2	3	⅜-in. gravel.	37.5	170	2,060	....	2,857,000	1,846,000	....
26	1	2	4	⅜-in. gravel.	42.9	171	1,700	....	2,308,000	....	....
27	1	3	6	⅜-in. gravel.	80.0	169	580	1,200,000	....	....	....

ing the deformations. The deformations were read in ten-thousandths of an inch simultaneously on each side of the specimen. The gaged length was 18 ins. in tension and 12 ins. in compression.

In the tension tests readings were made at each load increment of 250 lbs. on the specimen, which was equivalent to about 7 lbs. per sq. in. in cross-section. Details of the tension tests are given in Table L.

In compression tests, readings were taken at intervals of 5,000 lbs., equal to about 140 lbs. per sq. in. of section, except at the start, when readings were made at 250 lb. loads.

Prof. Bach's method of finding the true elastic curve was followed, and three curves plotted for each test; first, the curve representing the total deformation; second, the curve of sets, and third, the true elastic curve obtained by subtracting the second curve from the first, as shown in Fig. 66.

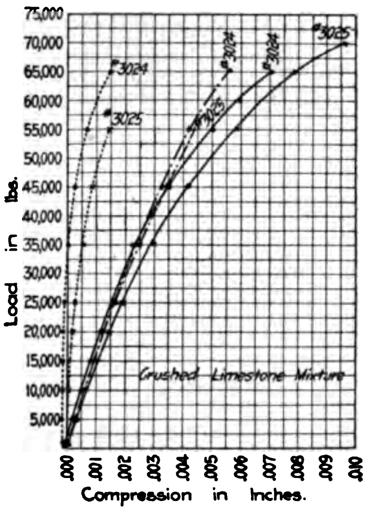


Fig. 66.—Stress Strain Diagram.

For each test the coefficient of elasticity, E, was calculated for three points on the elastic curve for each specimen. Table LIII. gives values of the moduli for a number of compression tests.

A fair idea of the value of the coefficient of elasticity is obtained from this table, although the values appear to be somewhat higher than those given in Tables XLV., XLIX. and

LI. The tendency of the value to decrease as the load increases should be noted. The values of E, as here shown, do not appear to be affected by the ultimate strength.

An average value of E for the crushed limestone, with the broken limestone mixture, as shown by this set of experiments, would be about 5,000,000 lbs. per sq. in. for both tension and compression, and for the sand and broken limestone mixtures about 3,600,000 lbs. per sq. in. in compression and 4,500,000 lbs. per sq. in. in tension. These values are based on too small a number of tests to be considered as more than indicating what the probable value of the coefficient is for large concrete specimens, made from full-sized stone. They are, however, of value when considered in connection with other tests here given. Many more tests are needed to determine more definitely the real value of the coefficient of elasticity.

TABLE LIII.—MODULI OF ELASTICITY (E) FOR COMPRESSION.

(Woolson's Tests.)

1:2:4 Concrete. Specimens 6 × 6 ins.

(Elastic Curve = Total Deformation Curve Minus Set Curve.)

Crushed Limestone and Broken Limestone.

Test No.	E at 135 lbs. per sq. in.	E at 682 lbs. per sq. in.	E at 1,227 lbs. per sq. in.	Breaking Load. lbs. per sq. in.
3018	5,600,000	5,600,000	5,600,000	3,400
3019	3,306,000	4,132,000	4,380,000	2,040
3020	5,540,000	5,890,000	4,676,000	2,120
3024	5,460,000	4,820,000	4,610,000	2,160
3025	5,560,000	5,208,000	4,287,000	2,190
3051	6,810,000	6,290,000	6,130,000	2,580
3062	5,515,000	4,140,000	2,980,000	1,455
Average..	5,398,900	5,154,300	4,666,200	2,278

Sand and Broken Limestone.

Test No.	E at 135 lbs. per sq. in.	E at 682 lbs. per sq. in.	E at 1,227 lbs. per sq. in.	Breaking Load. lbs. per sq. in.
3021	2,755,000	2,505,000	....	1,500
3022	4,130,000	3,940,000	3,380,000	1,920
3023	2,800,000	3,650,000	3,360,000	2,000
3026	4,100,000	3,725,000	3,210,000	1,850
3027	2,755,000	2,666,000	2,520,000	1,760
3058	4,190,000	4,190,000	3,350,000	1,672
Average..	3,687,000	3,446,000	3,164,000	1,784

Limiting Stresses to be Used in Choosing Modulus for Computations.—Inasmuch as the value of the coefficient of elasticity decreases as the unit stress increases, the question at once arises, what value of the coefficient should be used in the computations? It seems rational that a value of the modulus for the usual allowable stress, say from 0 to 600 lbs. per sq. in., should be used, it being understood that the unit stresses are to be kept within these limits. If a computation is desired for the ultimate strength, a value of the modulus at the ultimate strength of the concrete should be employed.

The coefficient of elasticity increases as the richness, and hence the strength, of the concrete increases. A study of Rafter's tests seems to indcate that the coefficient of elasticity is some function of the compressive strength. Falk, after a careful comparison of a large number of tests, states that there appears to be a direct relation between the coefficient of elasticity and the compressive strength, that concrete in compression seems to have a point that may be called the elastic limit, and its value is between one-half and two-thirds the ultimate strength. He concludes that up to this elastic limit the compressive coefficient of elasticity may be taken at 1,325 times the ultimate crushing strength. Or, expressed algebraically, we have:

$$E\,c = 1{,}325\ f\,c.$$

This formula gives slightly higher values than Thacher's formulas, which are derived from Kimball's tests, but agree more closely than the latter with more recent tests, which seem to indicate a higher coefficient than earlier ones. Thacher's formulas are as follows for concrete:

THACHER'S FORMULAS FOR COEFFICIENT OF ELASTICITY.

$$\text{7 days old} \quad E = 2{,}600{,}000 - 700{,}000\left(\frac{\text{volume of sand}}{\text{volume of cement}} - 2\right)$$

$$\text{1 month old} \quad E = 2{,}900{,}000 - 300{,}000\left(\text{do.} - 1\right).$$

$$\text{3 months old} \quad E = 3{,}600{,}000 - 500{,}000\left(\text{do.} - 2\right).$$

$$\text{6 months old} \quad E = 3{,}600{,}000 - 600{,}000\left(\text{do.} - 3\right).$$

If the term $\left(\frac{\text{volume of sand}}{\text{volume of cement}} - c\right)$ is zero or becomes a negative quality, the entire term is to be considered zero.

Table LIV. shows values of the modulus for different mixtures derived from the above formulas, the values of compressive stress used in Falk's formula being taken from Thacher's formulas, given on page 192. The values given in the last two columns of the table are average values and may be safely used in computations if desired.

Values given in the above tables are for loads taken from 0 to 600 lbs. per sq. in. If a higher loading than 600 lbs be used the values of the coefficient will be materially reduced. It has been found that for loads from 1,000 to 2,000 lbs. its values will be from $^2/_3$ to ½ that given above for loads not exceeding 600 lbs. per sq. in.

TABLE LIV.—SHOWING MODULI OF ELASTICITY OF CONCRETE.

Mixture.	Thacher's Formulas. 1 month.	Thacher's Formulas. 3 months.	Falk's Formulas. 1 month.	Falk's Formulas. 3 months.	Average Value. 1 month.	Average Value. 3 months.
1:1:3	2,900,000	3,600,000	3,740,000	4,450,000	3,200,000	4,000,000
1:2:4	2,600,000	3,600,000	3,180,000	3,980,000	2,800,000	3,800,000
1:2½:5	2,450,000	3,350,000	2,950,000	3,640,000	2,600,000	3,500,000
1:3:6	2,300,000	3,100,000	2,720,000	3,540,000	2,400,000	3,200,000
1:3½:7	2,150,000	2,850,000	2,480,000	3,230,000	2,200,000	3,000,000
1:4:8	2,000,000	2,600,000	2,250,000	3,930,000	2,000,000	2,800,000

The coefficient of cinder concrete differs somewhat from that of stone concrete. Table LV. gives the average values of a series of tests and is taken from the Watertown Arsenal Report of Tests for Eastern Expanded Metal Co. for 1898. Tests were in 12-in. cubes 3 months old:

TABLE LV.—ELASTIC PROPERTIES OF CINDER CONCRETE.

(From Watertown Arsenal Tests of 1898.)

12-inch Cubes. Age 3 Months.

American Portland Cement.	Proportion Cement.	Proportion Sand.	Proportion Cinder.	Modulus of elasticity per sq. in., between loads of 100-600 lbs.	Modulus of elasticity per sq. in., between loads of 100-1,000 lbs.	Modulus of elasticity per sq. in., between loads of 1,000-2,000 lbs.	Compressive strength per sq. in.
Alpha	1	1	3	2,500,000	2,500,000	1,429,000	2,780
Alpha	1	2	5	1,087,000	957,000	....	1,402
Alpha	1	2	5	1,471,000	1,286,000	....	1,715
Atlas	1	1	3	4,167,000	3,214,000	1,190,000	2,368
Atlas	1	1	3	2,083,000	1,875,000	1,351,000	2,580
Atlas	1	2	5	1,190,000	849,000	....	1,200
Atlas	1	2	5	1,087,000	865,000	....	1,263

Table LVI. gives Henby's tests for the compressive strength and the coefficient of elasticity of cinder concrete:

TABLE LVI.

(Henby's Tests for Cinder Concrete.)

Mixture.	Age, days.	Compression stress.	Modulus of elasticity.
1:2:4	30	1,006	1,461,000
1:2:5	30	823	1,396,000
1:3:6	30	500	1,225,000
1:2:5	60	700	1,330,000
1:3:6	60	640	916,000

Table LVII. gives additional tests made at Watertown Arsenal in 1903:

TABLE LVII.—ELASTIC PROPERTIES OF CINDER CONCRETE.
(From Watertown Arsenal Tests for Eastern Expanded Metal Co., 1903.)
Lehigh Portland Cement. 12-inch Cubes set in air.
Gage Length 5 inches.

Proportions.				Modulus of elasticity per square inch between loads of		
Cement.	Sand.	Cinder.	Age.	500 to 1,000 lbs.	At ultimate strength.	Compressive strength per square inch.
1	2	4	38	1,786,000	1,136,000	1,950
1	2	4	38	1,923,000	1,136,000	2,050
1	2	4	224	1,471,000	1,087,000	2,600
1	2	4	224	1,563,000	463,000	2,500
1	2½	5	38	1,250,000	....	1,400
1	2½	5	38	893,000	....	1,400
1	2½	5	224	1,136,000	893,000	1,980
1	2½	5	224	1,250,000	694,000	2,020
1	3	6	34	781,000	....	1,200
1	3	6	34	1,000,000	....	1,330
1	3	6	220	1,000,000	694,000	1,720
1	3	6	220	735,000	463,000	1,560

For mixtures of 1:2:3 or richer, it will probably be safe to use a modulus of 1,500,000 and for mixtures not leaner than 1:3:6, a value of from 800,000 to 1,000,000.

Authorities differ considerably on this question.

The proper value of the coefficient of elasticity to be used wil. depend upon the safe working stresses chosen, the mixture used, and the particular theoretical formulas employed in the computations. The values given above for different conditions will enable the engineer to choose a proper value for his computations.

CHAPTER XI.

PHYSICAL PROPERTIES OF REINFORCING METALS.

Metal is a most important factor in reinforced construction, as dependence is put upon it alone to care for all dangerous stresses. The manner in which it is used and the different forms employed will be explained in the succeeding chapters, but it is necessary before discussing the methods of determining the sectional area of metal needed for a given reinforced concrete member to take up in detail its physical properties, as it is necessary to understand them in order to make a proper choice for the reinforcement.

Until quite recently, in Europe, wrought iron was used exclusively for reinforcement, and is still in great favor, although steel is gradually replacing it. Wrought iron possesses many valuable characteristics, among which not the least is its property of being easily and safely welded. Steel possesses greater strength than iron and will, on this account, give greater economy if a high grade of concrete be used. The cost of iron and steel in Europe is about the same; in the United States, however, steel is the cheaper, and is used exclusively for reinforcement.

Wrought Iron.—When employed for reinforcement, iron is most frequently used in the form of round and square rods and flat bars; these, being of a recognized standard commercial quality, are easily obtained. The iron should be of good quality, with a breaking strength of about 50,000 lbs. per sq. in., and an elastic limit of at least one-half the ultimate strength, and should have an elongation of from 8 to 12 per cent. in a length of 8 ins. A bar should bend when cold 180° around a curve whose diameter is twice the diameter of the test piece without evidence of failure.

The unit breaking strength of iron increases as the section of the rod decreases, but the cost of the metal increases as the size becomes smaller.

Steel.—As has been stated, steel is used exclusively for reinforcement in America. This is undoubtedly because it is cheaper than wrought iron. Unfortunately authorities differ as to the quality of steel to be used for reinforcement, soft, medium and high steel being used by different engineers.

If a steel of good quality be employed, it is immaterial which be used, as first class structures have been built with each. However, soft and medium steel are better fitted for some classes of structures than high steel, while in others the high steel will answer just as well, with greater economy.

Open hearth steel is preferable to Bessemer steel, as it is more uniform in quality and does not possess the brittleness sometimes met with in Bessemer steel, and which makes the latter at times extremely dangerous for use as a reinforcing material. Fortunately it is possible to secure such an excellent quality of either soft or medium steel in the open markets, which is manufactured and sold under standard conditions, that an engineer can feel sure of the safety of his structure without the expense of exhaustive tests.

Open hearth steel, either acid or basic, should conform to the following requirements: The maximum limit of phosphorus in the finished material should not exceed .07% for acid and .05% for basic open hearth steel. Soft steel should have an ultimate strength of from 54,000 to 62,000 lbs., and an elastic limit of not less than one-half the ultimate strength; it should elongate 25% in 8 ins., and bend cold 180° flat on itself without fracture on outside of bend.

If medium steel is used, it should have an ultimate tensile strength of from 60,000 to 68,000 lbs. per sq. in., an elastic limit of not less than one-half the ultimate strength and should elongate not less than 22% in 8 ins., and bend cold 180° around a diameter equal to the thickness of the piece tested, without fracture on outside of bend.

In the above bending tests for soft and medium steel, the quality of the metal should be such that it shall stand the above described tests upon a test piece at least $^5/_{16}$-in. in diameter after being heated to a cherry red and cooled in water at a temperature of 70° F.

High steel, that is, steel containing a high percentage of carbon, is used for reinforcement by some engineers. Brittleness is to be feared in high steel, although this quality is not so dangerous when the metal is used in reinforced concrete as when used in heavy beams or shapes, as the concrete to a large extent absorbs the shocks and protects the steel. As a rule, a satisfactory product cannot be obtained on the open market and, unless the quantity of metal desired is large enough to warrant unusual precautions and careful inspection during manufacture, it will be almost impossible to secure a satisfactory material. The expense necessary to warrant a careful inspection will so increase the cost of the material that little or no economy will result from the use of the high steel, even though it possess 50 per cent. greater strength than medium steel. When it is desired to use a high steel it should contain little or no impurities, not more than .06 per cent. of phosphorus, not more than .06 per cent. of sulphur, and not less than 0.4 per cent or more than 0.8 per cent. of manganese and should contain from 0.5 to 0.6 per cent. of carbon. It should possess an ultimate tensile strength of at least 100,000 lbs. per sq. in. and an elastic limit of not less than one-half the ultimate strength, and should elongate not less than 10 per cent. in 8 ins. for a test piece from 3/8 to 3/4 in. in diameter. A test piece 1/2 in. in thickness should bend 110° without fracture around a diameter equal to its thickness.

In the design of steel structures it has been the custom to base the allowable working stress upon the ultimate strength of the steel, but in reinforced concrete design it is more rational to base the working stress upon the elastic limit of the steel. This practice has been adopted by the majority of engineers. The allowable working stress should fall well within the elastic limit, for as soon as this limit is reached the metal stretches rapidly and, decreasing in section, is loosened from the concrete, thereby destroying the monolithic action existing before this point is reached. As the metal stretches the concrete cracks badly and its usefulness as a structural material becomes impaired and is ultimately destroyed.

Other things being equal, the steel having the highest elastic limit will be the most satisfactory for reinforcement. Unfortunately steel having a high elastic limit has approximately the same coefficient of elasticity as low steel. The resistance of steel to deformation depends upon its coefficient of elasticity. The elongation suffered by the metal will not be proportional to its elastic limit, but to its coefficient of elasticity. Thus a steel with an elastic limit of, say 30,000 lbs. per sq. in., will at this limit stretch about 0.0010 of its length, while a steel with an elastic limit of, say, 50,000 lbs., will stretch about 0.00167 times its length at its elastic limit. The first limit, 0.0010, is, as is explained in Chapter XVIII., about the maximum stretch allowable in reinforced concrete work, hence little will be gained by the use of the high steel. However, a higher factor of safety will result when the high steel is used, and the working stresses may more easily approximate those developed at the allowable limit of stretch, and a real economy secured by the use of high steel. When high steel of a satisfactory quality can be secured at a price not greatly in excess of that of medium steel, considerable saving may result. Thus a saving of as much as 25 per cent. over mild steel may ensue if the high steel rods be secured, as is often the case, at an advance of about 10 per cent. in price over mild steel.

The elastic limit or yield point of ordinary mild steel varies from 30,000 to 40,000 lbs. per sq. in.; 36,000 lbs. may be taken as a fair average. The elastic limit of high carbon steel varies from 50,000 to 60,000 lbs. per sq. in.; 54,000 lbs. may be taken as a safe working value.

If the high steel be used and it is assumed that the additional stretch in the concrete is not injurious, a much smaller percentage of steel will be needed to secure the same moment of resistance than when mild steel is used.

The question in regard to whether or not the minute cracks in the bottom side of the beams will prove injurious and permit the corrosion of the steel, is one requiring careful investigation. When data on this subject become available, many vexing questions concerning reinforced concrete will be solved. In the meantime it would seem to be conservative practice to limit the stress in the steel so that the cracks cannot prove dangerous in as far as the economy of reinforced construction permits.

When the concrete is to be subjected to a moist atmosphere or to corrosive gases, steel with lower working stresses should be employed or a different form of construction adopted.

In many classes of structures there is no doubt high steel may be used with economy and without in any way endangering the structure. Thus in walls and floors in buildings not subject to shocks or vibrations, retaining walls, reservoirs, etc., not subjected to severe conditions, the high steel will, in the majority of cases, prove satisfactory. For railway bridges, factory and warehouse floors, subjected to vibration, shocks, etc., a soft ductile steel should in all cases be used. Where soft steel is used its great ductility will enable it to stretch and give warning of failure long before final rupture takes place.

Cost of Reinforcement.—A statement of the comparative cost of a number of the special reinforcing bars in general use would be of interest. The cost of steel, however, fluctuates from time to time, and prices are based on an average price per pound of plain steel rods in 50-ton lots at the mills. The standard size of rod on which a base price is assumed is ¾ in. or over. For sizes below ¾ in. the price increases, according to the schedule of the Associated Steel Manufacturers.

Table LVIII. gives the relative prices for different sizes of plain rods, assuming the price of ¾-in. plain Bessemer rods at $30 a ton, or 1.5 cts. per pound. The cost of deformed rods is from $8 to $12 a ton more than the cost of plain rods, depending upon the condition of the market. Thus, the market price for a ¾-in. deformed bar of O. H. steel, at an increase of $8 per ton, will be 1.65 + 0.40 = 2.05 cts. per pound.

For Ransome twisted bars about $4 a ton should be added to the cost of plain bars for twisting.

For the Kahn bar, owing to its peculiar form, more metal must be used for equal strength than when plain or ordinary deformed bars are used. Hence, for equal pound prices the cost of the Kahn reinforcement will be higher. It should, however, be remembered that a large portion of this extra metal is used for stirrups. The manufacturers do not supply a schedule of prices for different sized bars, but quote special prices for each job upon which they bid.

TABLE LVIII.—COST OF DIFFERENT SIZES OF PLAIN RODS

(Price in cents per pound on basis of 50-ton lots at mill.)

Size.	Bessemer rods, plain.	Open hearth.
¾ to 3 inch	1.50	1.65
⅝ to 11/16 "	1.60	1.76
½ to 9/16 "	1.70	1.86
7/16 "	1.90	2.07
⅜ "	2.00	2.18
5/16 "	2.10	2.28
¼ to 9/32 "	2.20	2.39

The cost of putting in steel for retaining walls, arches and ordinary constructions varies from \$5 to \$8 per ton. In building work it may run up to \$15 a ton.

There are a number of firms furnishing reinforcements for beams and girders fabricated into units or trusses ready to put into place. Some of these make special provision for attaching the slab reinforcement. This method of reinforcement, from experience up to date, has been found to cost from 33 to 50 per cent. more than properly constructed single-bar systems.

CHAPTER XII.

PRINCIPLES AND DISPOSITION OF REINFORCEMENT.

The development of reinforced concrete as a distinct system of construction dates from the time that the function of the metallic reinforcement was understood. The highest success in the use of this form of construction is attained only when a maximum strength is secured at a minimum cost. This is possible only by using the proper proportions of the two elements—concrete and metal—these being placed in such a manner as to obtain the greatest strength.

Certain fundamental principles are essential to this system of construction. When a solid body is acted upon by external forces, stresses are produced in its interior, which tend to change its shape. These stresses may in general be resolved into three kinds, viz.: tension, compression and shear. The materials of which reinforced concrete is composed behave differently, according to the nature of the stresses brought upon them.

The metal, iron or steel, resists these three kinds of stresses equally well. Cement concrete, however, while acting well under compression, offers comparatively small resistance to tension and shear. These facts form the basis for construction in reinforced concrete; for by providing an ample section of concrete to resist the compressive stresses and strengthening that part of the given structure where dangerous tensile and shearing stresses are developed by incorporating in the body a metal skeleton, a safe and economic structure is obtained. At times metal is also used to reinforce the part under compression, but it is evident that the greatest utility is obtained when the only function of the steel is to take care of the tension.

Many engineers neglect the tensile strength of concrete and rely entirely on the reinforcement to care for tensile stresses. While this is on the side of safety, it is not always wise to neglect the tensile strength of the concrete.

In order that this heterogeneous mass of reinforced concrete may act as a unit, it is necessary that the internal stresses shall be transmitted from the concrete to the metal. That stability may be assured, it is requisite that the shearing stress developed at the surface of contact between the two materials shall not be greater than the adhesion between them. When the adhesion is less than the shear it is necessary to use some form of mechanical bonding.

Classification of Reinforced Concrete Members.—Reinforced concrete pieces may be classified according to the manner in which they are used. This is determined by the form of the piece and the method of application of the external forces.

Straight pieces, viz., pieces having a rectangular longitudinal section, may be strained in compression and flexure. Pieces having a curved longitudinal section may be strained in flexure, compression and tension. Pieces strained in flexure are in general subjected to shearing stresses. This will, therefore, not modify the following classification. We will divide reinforced concrete pieces into the five following classes:

First: Straight pieces strained in flexure, as beams.

Second: Curved pieces strained in flexure, as arches.

Third: Straight pieces strained in compression, as columns.

Fourth: Curved pieces strained in compression, as pipes subjected to external pressure.

Fifth: Curved pieces strained in tension, as pipes subjected to internal pressure.

Reinforced concrete is not adapted to straight pieces strained in tension.

Straight Pieces—Beams and Slabs.—Definition: If the external forces act normal to the axis of the rectangular piece, it will be strained in simple flexure; if obliquely, in composite flexure. All cases of composite flexure may be resolved into compression and simple bending. Only in special cases of composite flexure, which occur very rarely, when the compression is excessive, will it be necessary to modify the character of the reinforcement. Under such circumstances the piece should be treated as a straight piece subject to compression. A large variety of structures may be grouped under this classification. Straight pieces are of two kinds, slabs and beams. Sometimes slabs and beams are used together, and we then have ribbed slabs or T-beams. Reinforced concrete beams may have any one of the forms which are ascribed to beams.* Slabs are usually of three forms. They may rest simply upon the supports, be fixed at the supports or be continuous over them. In the analysis of stresses, sections of a slab may be treated as a shallow beam.

Flexural Stresses.—In a beam supported at both ends and acted upon by a force normal to its axis, it is evident that at any vertical section, by virtue of the bending stress, that part of the beam above the neutral axis is under compression and that part below under tension. Both of these stresses attain maximum values at the outermost fibres of the beam and decrease to zero at the neutral axis. The intensity of this stress at any point may be obtained from the well-known equation of flexure:

$$f = \frac{Mc}{I},$$

when M represents the bending moment, $c =$ the shortest distance from the given point at the neutral axis; $I =$ the moment of inertia of the given beam, and $f =$ the intensity of the stress at the given point.

The bending moment M, and with it the intensity of the horizontal stress f, increases from the end toward the middle of the beam. Thus we see that the horizontal stress f varies, not only in a vertical direction on both sides of the neutral axis, but also in the direction of the length of the beam.

If we consider the beam as composed of a series of horizontal layers, this increase of horizontal stress from one layer to the next develops a force tending to slide one longitudinal layer past the one next above. This force is called longitudinal or horizontal shear.

Vertical Shearing.—The vertical shear at any section of a beam is the reaction due to the load at one end minus that part of the load lying between the end and girder section.

Owing to the low shearing stress of concrete, this should not be neglected in proportioning reinforced concrete pieces.

In order that there be equilibrium in a beam, the summation of the internal stresses must be equal to zero. By well-known methods of analysis the direction and intensity of the stresses in a beam may be obtained. Fig. 67 shows the lines of stresses in a beam under flexure as given by Rankine.

*See Merriman's Mechanics of Materials, Chaps. V., VI. and VII., 10th edition, 1906.

Tensile and shearing stresses tending to rupture always exist in a concrete piece strained in flexure. In placing the metal rein-

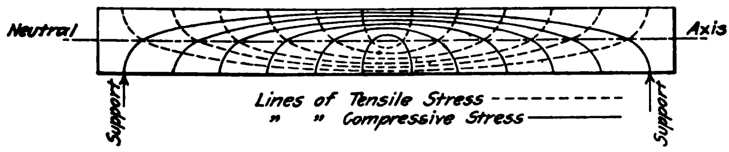


Fig. 67.—Lines of Stress in a Beam Under Flexure.

forcement in a concrete piece, both the tensile and shearing stress should be taken into consideration.

Disposition of the Reinforcement.—It is evident that in order to

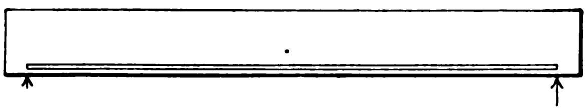


Fig. 68.

utilize the maximum strength of the reinforcement, it should be placed as near as possible to the outer fibre of the piece under tension.

Let us first consider a simple beam loaded from above and sup-

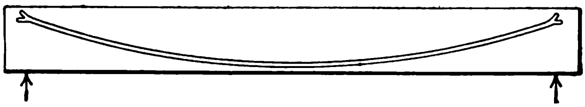


Fig. 69.

ported at two points. Tensile stresses will be developed in the lower part of the beam throughout its length. To care for these the reinforcement should be placed in the bottom of the piece, as

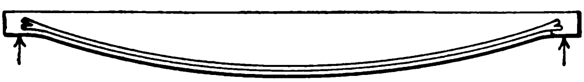


Fig. 70.

near as possible to the lower face, and extend over the entire portion between the supports. The reinforcement may be straight, as in Fig. 68. This is the simplest form of reinforcement. It may be given a curved form, as in Fig. 69, since the bending moment and the tensile stress increase from the supports to the middle of the beam. This curved form should be compared with the curve of tensile stresses shown in Fig. 67. When the curved form is used

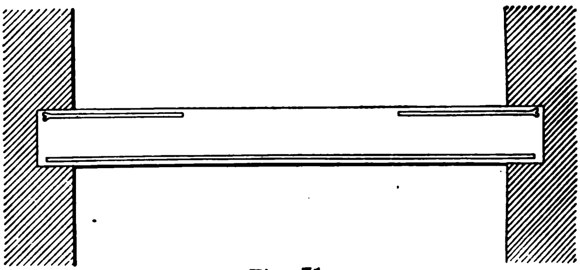


Fig. 71.

the lower face of the beam sometimes follows the curve of the reinforcement, as shown in Fig. 70. When this form is used, care

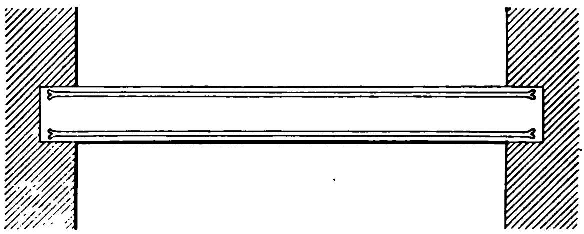


Fig. 72.

must be taken not to reduce the section near the supports so that it will be unable to carry the end shear.

When the beam is fixed at the ends, as shown in Fig. 71, the

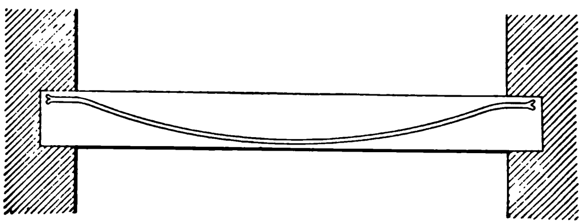


Fig. 73.

bending moment changes in character between the supports. In the middle portion the lower part of the beam is in tension—near the supports the upper part is in tension. A straight bar extending over the entire length of the bottom and two short bars at the top, extending over the region of tension, and anchored at the supports, provide one form of reinforcement for this form of beam.

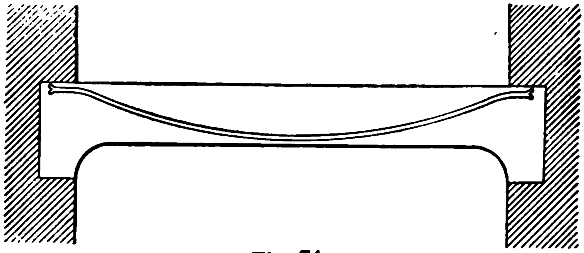


Fig. 74.

As the region of tensile stresses is somewhat indeterminate, the top reinforcement is sometimes made to extend throughout the length of the entire piece, as shown in Fig. 72.

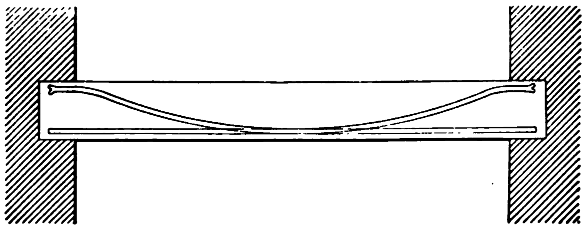


Fig. 75.

This gives us the type of double reinforcement. A single curved reinforcement, as shown in Fig. 73, may be used, extending along the lower part of the middle of the beam and rising to the upper part at the support, giving the desired resistance, both

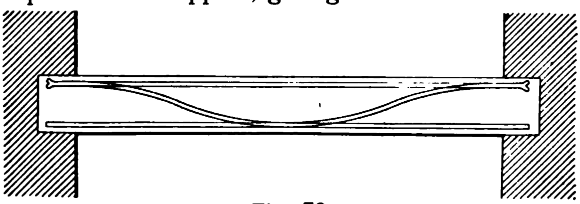


Fig. 76.

at the center and the ends. A modification of the form of the piece by thickening the beam at the ends (Fig. 74), gives additional strength at the supports. By adding a straight bar at the bottom to the curved bar we get the mixed system shown in Fig.

75. Another straight bar at the top may be added, and we obtain the form shown in Fig. 76.

The upper reinforcements of Figs. 72 and 76, while stressed in tension at the ends, strengthen the upper middle portion of the beam under compression by its resisting value in compression.

The forms of reinforcement described above are the more simple dispositions. By the addition of more rods, many complicated systems may be evolved, but the action of the bars to resist tension can be in the last analysis reduced to one of these forms.

CHAPTER XIII.

MECHANICAL BOND.

Longitudinal Shearing Stresses.—As has been explained, there are two kinds of shearing stresses which must be considered in reinforced concrete construction: (1) The shearing which is developed at the surface of contact between the concrete and the metal, (2) the vertical shear, existing in the mass of concrete, which may be defined as the algebraic sum of all the vertical components of the internal stresses at any section of the beam.

It is essential that both kinds of shear be taken into account when proportioning reinforced concrete pieces, for in the case of the first the safety and utility of this form of concrete construction depends upon the adhesion being ample to care for the horizontal shear, and as in other forms of construction, if ample provision is not made for the second, the structure will fail. The question of adhesion and safe values for horizontal shear is discussed in another chapter.

It is our purpose here to take into consideration the methods of caring for this shear where the adhesion between the two materials is not sufficient, or when it is not thought wise to rely entirely upon it.

Many engineers hold that some form of mechanical bonding should always be used; others that it is not necessary. In Europe until very recently reliance was placed upon adhesion alone to care for the horizontal shear. In America mechanical bonding is in great favor. It may be conservatively stated that where proper shearing values are used, reliance may safely be placed on the adhesion, but there can be no objection and it is often desirable to supplement it by some form of mechanical bonding.

Mechanical bonding may consist of deformed rods, supplementary rods, stirrups or anchors, all intended to make a closer union between the reinforcement and the concrete. This bonding, acting in conjunction with the concrete, may be considered as fulfilling the functions of the web of a metal beam.

Composite Bars.—One of the first forms of mechanical bonding which deserves mention was invented by Thaddeus P. Hyatt. It consists in passing short horizontal rods through holes drilled or punched in the main reinforcing bar, as shown in Fig. 77.

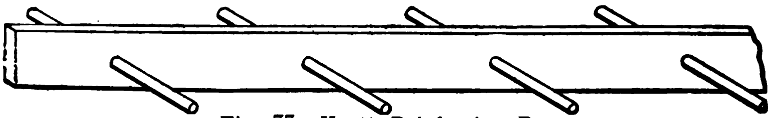


Fig. 77.—Hyatt Reinforcing Bar.

When a series of parallel reinforcing bars is used, the small rods may be increased in length and pass through a number of the series. This gives a form shown in Fig. 104, under floor slabs.

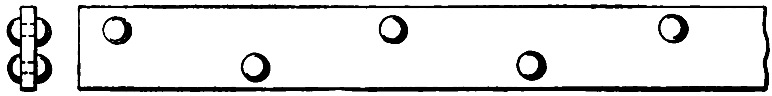


Fig. 78.—Thacher Riveted Bar.

Either square or flat bars may be used. In the construction of bridges, Mr. Edwin Thacher formerly used flat bars of large section, and replaced the rods with rivets, the heads of which give

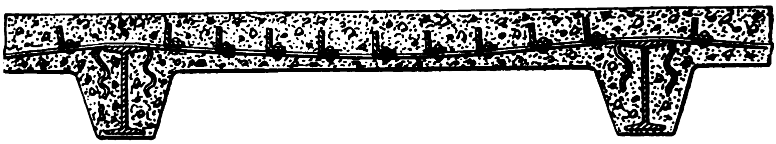


Fig. 79.—Klett Reinforcing Bar.

the necessary bond, as shown by Fig. 78. An objection to the use of bars of this kind is the reduction of area due to the drilled or punched holes.

In the Klett system (Fig. 79) angles of a length equal to the

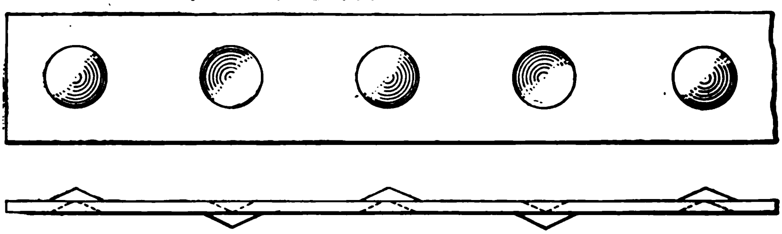


Fig. 80.—Staff Indented Bar.

width of a flat bar are riveted at intervals throughout its length. This system is used for floor slabs between beams.

Indented Flat Bars.—In the Staff system (Fig. 80) a flat bar is used, with alternating projections and depressions of circular form, stamped in the metal when it is rolled.

Twisted Steel Bars.—Steel bars twisted cold are the invention of Mr. E. L. Ransome. Square bars are usually employed, but the

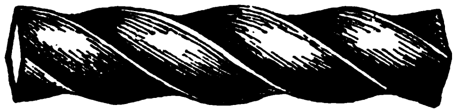


Fig. 81.—Ransome Twisted Bar.

system may be applied to a variety of sections. The twisted form (Fig. 81) gives a firm grip upon the concrete, thereby greatly assisting the adhesion between the two materials. These bars have been very widely used in various Ransome concrete steel constructions. They may be purchased already twisted, or plain bars may be bought and twisted at the site of the work. The apparatus necessary for twisting is inexpensive, and one man can easily twist the rods as fast as needed. The accompanying table gives approximately the number of twists per foot of bars:

Size in inches.	Number of twists.	Size in inches.	Number of twists.	Size in inches.	Number of twists.
1/8 × 1/8	7	1 × 1	1	1 3/4 × 1 3/4	1/8
1/4 × 1/4	5	1 1/4 × 1 1/4	1/2	2 × 2	1/10
1/2 × 1/2	3	1 1/2 × 1 1/2	3/8	.....	...

The operation of twisting the rods cold modifies the qualities of the metal, raising the elastic limit and ultimate strength, as shown by the following table:

Size of Bars, inches.	Elastic Limit.		Ultimate Strength.	
	Plain.	Twisted.	Plain.	Twisted.
1/4 × 1/4	46,170	62,350	57,800	86,700
3/4 × 3/4	34,840	56,720	59,200	85,240
7/8 × 7/8	39,700	56,150	62,100	84,730

De Man Bars.—The twisted bar shown in Fig. 82 is the inven-

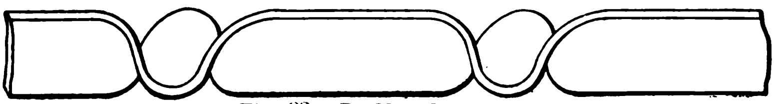


Fig. 82.—De Man Crimped Bar.

tion of Mr. Alphonse De Man. It is intended for use in cinder concrete for floor slabs. This bar is used in sizes varying from 1-10 to 1/4 in. in thickness and from 1/4 to 1 1/2 ins. in width; 1/8 × 3/4-in. bars have been most extensively used. At regular intervals along its length the bar is crimped so as to bring a portion of the normally vertical web into approximately a horizontal position and form a variation in shape which gives a mechanical bond. This bar is mostly used for floor slabs between steel beams, and when so used the ends are crimped to a horizontal position and are bent to hook over the flanges of the I-beams. This bar is used for spans up to 8 ft. It is set with the broad dimension vertical.

The Lug Bar.—This bar, as is shown in Fig. 82A, while similar to the Ransome twisted bar, has a hexagonal section with two short and two long sides and has lugs projecting at intervals from its short sides. The irregular section with projecting lugs is obtained by special rolling. Like the Ransome bar the lug bar is twisted cold. The purpose of the lugs is to assist the twisted section in securing a firm grip on the concrete and resist any tendency on the part of the bar to untwist under tension. The bars are rolled from mild steel

Fig. 82A. The Lug Bar.

with an elastic limit of not less than 32,000 lbs. and an ultimate strength of between 55,000 and 65,000 lbs. By cold twisting the ultimate strength is raised to about 84,000 lbs. per sq. in. and the elongation is much less than before. As a result yielding takes place slowly and uniformly nearly up to the ultimate strength without any sudden falling off of load carrying capacity. This bar is manufactured by The General Fireproofing Co., Youngstown, O. The sizes, weights and sections are given in the accompanying table. As will be seen, the bar is rolled in sections, corresponding to from ¼-in. to 1¼-in. square bars, and the section is free from sharp angles which might start cracks in the concrete.

PROPERTIES OF COLD TWISTED LUG BAR.

Size.	Approx. Wt. per foot.	Net sec. area.
¼"	.222	.0625
⅜"	.492	.1406
½"	.870	.2500
⅝"	1.350	.3906
¾"	1.940	.5625
⅞"	2.640	.7656
1 "	3.450	1.0000
1⅛"	4.350	1.2656
1¼"	5.370	1.5625
1½"	7.700	2.2500

The Cup Bar.—This bar, as shown in Fig. 82B, consists of a bar of round section having four longitudinal ribs connected by transverse ribs rolled upon its perimeter. These ribs form depressions or cups, which give a firm hold upon the concrete.

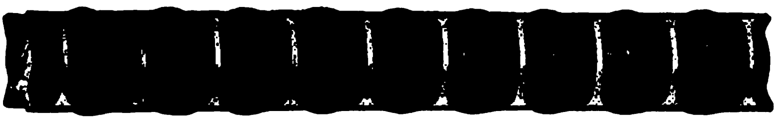


Fig. 82B. The Cup Bar.

This bar is manufactured in sizes from ⅜ in. to 1¼ in. varying by ⅛ in., the net area of each section being the same as a square bar of the same designation. It will be seen that the cross-section of the bar is absolute, no allowances have to be made for the deformations. Another point to be noted is that in this bar the fibers are not distorted from a straight line with the result that no injury to strength results from the deformations. In conclusion the rolling necessary to produce the cups has the effect of developing a more uniform and compact fiber structure than is produced in a plain bar as ordinarily rolled. The elastic limit and ultimate strength are increased and the ductility is not reduced. This bar is manufactured by the Trussed Concrete Steel Co., of Detroit, Mich.

Corrugated Bars.—Steel bars of square section, with corrugations on all four sides, as shown in Figs. 83 and 84, are the invention of Mr. A. L. Johnson, M. Am. Soc. C. E., and are manufactured by the St. Louis Expanded Metal & Corrugated Bar Co., St. Louis, Mo. Two forms are used; the old-style bar (Fig.

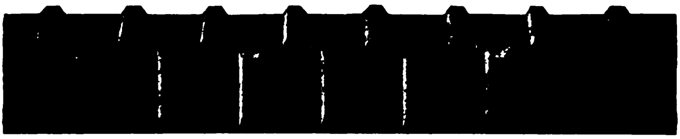


Fig. 83.—Corrugated Bar, Old Style.

83), which is preferred in some situations, and has some metal which is not available for trength, and the new bar (Fig. 84), which, it is claimed, has a constant cross-section and is the most economic in metal. Both bars are manufactured of soft, medium and high carbon steel. Another type of corrugated bar is shown

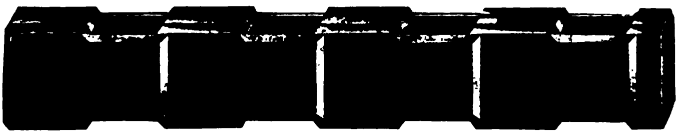


Fig. 84.—Corrugated Bar, New Style.

in Fig. 85; this bar is flat in section and may be used in situations where a flat bar is desirable.

The patent on this bar covers all bars which can be rolled having a ribbed surface in which the angle made by the sides of the ribs with a plane at right angles to the axis of the bar is less than the angle of friction between the concrete and the metal. These

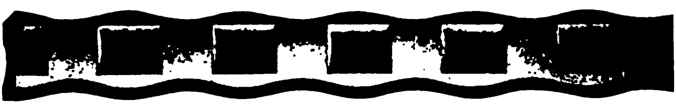


Fig. 85.—Corrugated Flat Bar.

corrugations are intended to supplement the adhesion which exists between the concrete and the metal, and should assist materially in resisting horizontal shear. Table LIX. gives the sizes, net section and weight of the three styles of corrugated bars described. All sizes are subject to a variation in weight of 5 per cent. either way.

TABLE LIX.—SIZES, SECTIONS AND WEIGHTS OF CORRUGATED BARS.

Old Bar			New Bar			Flat Bar		
Size in inches.	Net section, sq. in.	Weight per ft.	Size in inches.	Net section sq. in.	Weight per ft.	Size in inches.	Net section sq. in.	Weight per ft.
½	0.18	0.64	¼	0.06	0.24	¼ × 1	0.19	0.73
¾	0.37	1.35	½	0.25	0.85	5/16 × 1¼	0.32	1.18
⅞	0.55	1.95	⅝	0.39	1.33	⅜ × 1⅜	0.41	1.35
1	0.70	2.70	¾	0.56	1.91	⅜ × 1¾	0.54	1.97
1¼	1.07	4.00	⅞	0.77	2.60	⅜ × 2	0.65	2.27
			1	1.00	3.40	⅜ × 2½	0.80	2.85
			1¼	1.56	5.31			

While this bar may be rolled from medium or soft steel, the manufacturers use, almost exclusively, a high carbon steel, made from old steel rails. This steel has an elastic limit varying from 50,000 to 60,000 lbs. per sq. in., and a breaking strength of about 100,000 lbs. This value is from 60 to 70 per cent. higher than the elastic limit of mild steel, and on account of this high elastic limit only about $^7/_{10}$ as much metal is used as when mild steel is employed.

Thacher Steel Bars.—The Thacher bulbed bar is the invention of Mr. Edwin Thacher, M. Am. Soc. C. E., and is shown in Fig. 86. Thacher bars are manufactured by two methods, which are, respectively, designated as No. 1 mill and No. 2 mill bars. At No. 1 mill round bars are reheated and rerolled to the finished shape. This method is used for bars that are to be connected by sleeve nuts; 6 ins. at each end of the bar, on which thread is to be cut,

Fig. 86.—Thacher Bar.

being left round. Such bars are used in arches for which adjustments in the bars are very essential. They are also used when bars are required of a greater length than can be rolled or shipped in one length, and for all bars under ½ in. and over 1½ ins in diameter. No. 2 mill bars are rolled to the finished shape at the rolling mills without reheating, and are used when no splices or adjustments are necessary. No. 1 mill bars, varying by sixteenths, are rolled on special order.

Table LX. gives the properties of both kinds of bars.

TABLE LX.—GIVING AREAS AND WEIGHTS PER LINEAR FOOT OF THACHER AND DIAMOND BARS.

Nominal diameter, ins.	No. 1 Mill. Area, sq. in.	No. 1 Mill. Weight, per lin. ft.	No. 2 Mill. Area, sq. in.	No. 2 Mill. Weight, per lin. ft.	Diamond Bar. Area, sq. in.	Diamond Bar. Weight, per lin. ft.
1/4	0.047	0.16	...	...	0.0625	0.213
3/8	0.10	0.34	...	...	0.1406	0.478
1/2	0.18	0.61	0.17	0.58	0.25	0.85
5/8	0.28	0.95	0.27	0.92	0.39	1.33
3/4	0.41	1.39	0.39	1.34	0.56	1.91
7/8	0.55	1.87	0.53	1.79	0.76	2.60
1	0.71	2.42	0.68	2.32	1.00	3.40
1 1/8	0.90	3.06	...	...	...	...
1 1/4	1.10	3.74	1.04	3.55	1.56	5.31
1 3/8	1.32	4.49	...	...	...	...
1 1/2	1.56	5.30	1.53	5.20	...	...
1 5/8	1.81	6.16	...	...	...	...
1 3/4	2.08	7.07	...	...	...	...
1 7/8	2.35	8.00	...	...	...	...
2	2.65	9.02	...	...	...	...

In the majority of cases variation in rolling will not exceed 2½ per cent., but in some cases the variation from standard may reach 5 per cent.

The purpose of the inventor in designing these bars has been to avoid all sharp corners, knife edges and abrupt offsets, and all changes of section are made by gentle curves, thereby avoiding any tendency to split, crack or shear the concrete. These bars cannot slip in the concrete, and their peculiar form enables the concrete to be easily tamped about them without leaving any voids at or near the surface of contact. These bars are usually rolled from medium steel, but may, on special orders, be secured of soft or high carbon steel. For description and illustration of the Diamond bar see page 235A. The patents of the Thacher and Diamond bars are controlled by the Concrete-Steel Engineering Co., of New York.

Special Arrangement of Bars.—There are still other means of caring for the horizontal shear than by the use of deformed rods. The form which the reinforcement takes in the concrete may modify materially the amount of horizontal shear. By using some form of curved reinforcement the amount of tangental stress may be reduced and dangerous tendency to slipping avoided. Supplementary reinforcements in the form of rods, cross bars, straps and

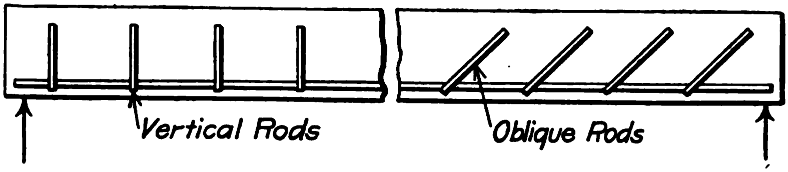


Fig. 87.

stirrups, normal to or at an angle with the main bar, and extending up into the concrete, are used as a means of uniting more closely the steel and concrete. The rods may or may not be attached to the main bar. The simplest form is shown in Fig. 87

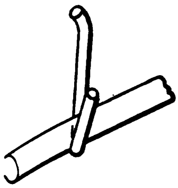


Fig. 88.

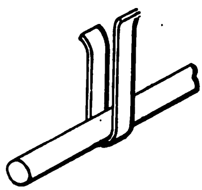


Fig. 89.

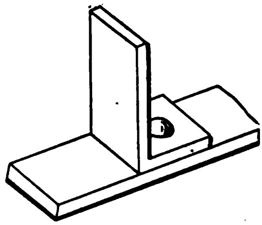


Fig. 90.

and consists of a number of independent vertical or oblique rods extending upward into the beam. These rods resist throughout their height the induced shear between the adjacent horizontal strata of the concrete. When fixed to the main bar they prevent

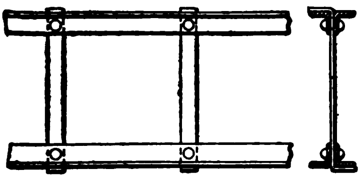


Fig 91.

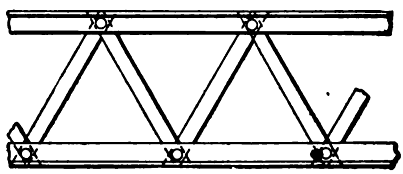


Fig. 92.

it from slipping in the concrete. These rods may have their ends simply hooked over the main rod (Fig. 88), may have the form of stirrups (Fig. 89), may be riveted to the main bar (Fig. 90) or may be struck up from the main reinforcement as in the Kahn bar (Fig. 93).

When supplementary rods of uniform section are used, it is rational to space them at increasing distances from the support in accordance with the law of variation of shearing stress. When these rods are used in conjunction with a single main reinforcement, the system may be considered as acting as a half truss, the steel furnishing the tension members and the concrete completing the truss and replacing the compression members. It is somewhat difficult to hold these secondary rods in position until the concrete is placed. On this account when a double reinforcement is not needed it is often the practice to use a light reinforcing bar in the compression flange. The supplementary rods are then riveted at both ends to the two bars, as shown in Fig. 91 and Fig. 92, thereby securing a light skeleton work or truss, which is easily held in place during the depositing of the concrete. When the light truss form is used it is sometimes made strong enough to support the concrete and the form until the former has set, after which the structure acts as a monolith.

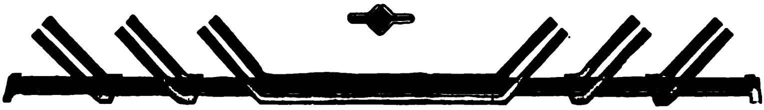


Fig. 93.—Kahn Trussed Bar.

The Kahn Trussed Bar.—This bar was invented by Mr. Julius Kahn, Assoc. M. Am. Soc. C. E. It furnishes a means of reinforcement in both the horizontal and vertical planes. It is in effect a stirrup system with the stirrups forming a part of the bar. This is accomplished by using a bar of the cross section, shown in Fig. 93, and shearing sections of the webs upward into an inclined position, on both sides of the main body. These bars are sheared in a number of different ways to suit varying conditions of loading. The bar is manufactured by the Trussed Concrete Steel Co., Detroit, Mich., in the following sizes:

Size.	Area, sq. ins.	Weight, lb. per ft.
1½" × ½"	3.38	1.4
2 3/16" × ¾"	0.78	2.7
3" × 1"	1.42	4.8
3¾" × 1¼"	2.00	6.9

It may be obtained of any desired length and sheared as the customer desires. These bars, with shallow web members, are used for slabs, but the best results are obtained when they are used in beams.

The Monolith Steel Bar.—This bar has grooves in its sides and a curved surface, as shown by Fig. 94. Stirrups are attached by placing the stirrup rods in the grooves and pressing the lips of the grooves together by means of a power press. The method of attachment is shown in Fig. 94. It is stated that the attach-

Fig. 94.—Monolith Reinforcing Bar and Stirrup.

ment is so rigid that the stirrup rods break without slipping, The sizes, sections and weights of this bar are given below. This bar is manufactured by the Monolith Steel Co., Inc., Washington, D. C.

Approximate size, inches.	Area, sq. ins.	Weight, lbs. per ft.	Size of stirrup.
$^{9}/_{16} \times \frac{5}{8}$	0.25	0.85	$^{3}/_{16}$
$\frac{7}{8} \times 1$	0.64	2.37	$^{3}/_{16}$
$1\frac{1}{8} \times 1\frac{1}{4}$	1.00	3.37	$\frac{3}{8}$
$1^{11}/_{16} \times 1\frac{7}{8}$	2.25	7.58	$^{9}/_{16}$

The Monolithic Steel Company give the following rules for spacing of web members in beams and girders:

If diagonal members are used, there must be five double members in each half of the span.

Let L be the length of half the span. Then the spacing, beginning at the end of the clear span, is as follows, measuring in each case from the end:

To the first web member	.07 L
" " second " "	.17 L
" " third " "	.30 L
" " fourth " "	.50 L
" " fifth " "	.78 L

If vertical members are used, there must be seven double members in each half of the span. The distance from the end of the clear span to the successive members is as follows:

To the first web member	.05 L
" " second " "	.12 L
" " third " "	.21 L
" " fourth " "	.31 L
" " fifth " "	.42 L
" " sixth " "	.55 L
" " seventh " "	.72 L

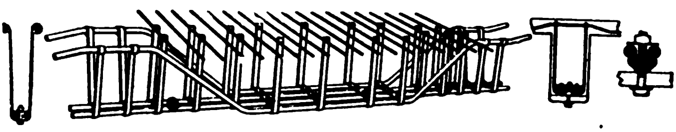


Fig. 95.—Unit Reinforcing Frame.

If the bearing of the beam is great enough, one or two additional web members should be spaced between the end of the reinforcing bar and the near edge of the clear span. Each reinforcing bar should be provided with web members as above indicated.

Reverse bars should also have, in each half of their length, the same number of web members as the main bars. The web

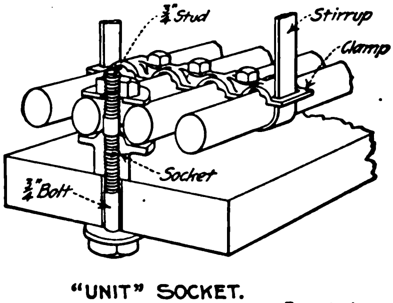


Fig. 96.—Unit Socket and Spacer.

members of reverse bars should be spaced so as to fall in the spaces between the web members of the main bars.

Unit Girder Frame.—This frame has the reinforcing rods and stirrups rigidly held together; Fig. 95 shows the arrangement of the different members. The rods are held together and in place by a unit socket. Figure 96 shows details of this socket. The patents covering the details of the unit frame and socket are controlled by Tucker & Vinton, New York.

The Scofield Spacing and Reinforcing Bars.—This bar is the invention of Mr. E. M. Scofield, Assoc. M. Am. Soc. C. E. The purpose of this bar is to take the place of stirrups in caring for longitudinal shear. In addition to this the bar automatically spaces and holds in place the main rods until the concrete is deposited and then performs the function of bonding together the rods and the concrete. This bar is manufactured by the Scofield Co., of Philadelphia, Pa. Figure 97 shows the form of this bar and the manner of its application to floor construction.

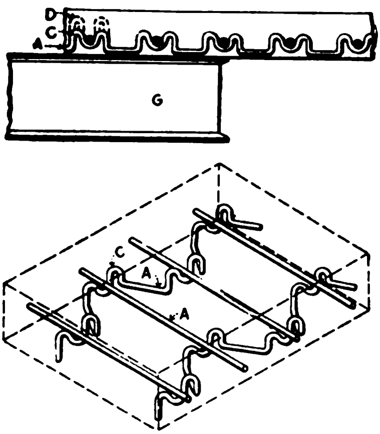


Fig. 97.—Scofield Spacing and Reinforcing Bar.

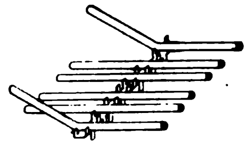


Fig. 98.— Cummings Spacing Chair.

Cummings Chair.—The Cummings chair is another device designed to space and hold the reinforcements in position, both vertically and horizontally, while the concrete is being placed and until it is set. The chair is stamped from strips of sheet steel and has regular projections, which are bent up to space the reinforcing rods and bent down to hold them the proper distance above the bottom of the forms. The projections are adjusted before the chair is placed in the forms and the steel reinforcement is then fixed in the required position. The chair is entirely surrounded by concrete and remains in place after the forms are removed. The character of the chair is shown in Fig. 98. These chairs are made in sizes suitable for any slab or beam; they are the invention of Mr. Robt. A. Cummings, M. Am. Soc. C. E., and are manufactured by the Cummings Structural Concrete Co., Pittsburg, Pa.

The Diamond Bar.—This bar is the invention of Mr. William Mueser, M. Am. Soc. C. E. It consists, as shown in Fig. 98A, of a round bar with spiral projecting ribs running around it in opposite directions. The pitch of the ribs running in one direction is the same as those running in the opposite direction, thereby preventing any tendency to turn when the bar is under strain. The spiral ribs meet on opposite sides of the bar two longitudinal ribs formed in the process of rolling. The ribs have a gentle slope, rounded off at top and base, and are made as high as is necessary to develop the full strength of the bar. This is obtained with practically no loss of metal and gives a bar of constant cross section.

Fig. 98A—The Diamond Bar.

The Maxwell Trussed Bar.—This bar consists of a specially rolled half round section with flanges extending from the edges of its flat side, to which are attached a trussing of wire to form a web reinforcement for a concrete beam. The attachment is obtained by punching out slots the size of the trussing wire, fitting the wire in the slots and upsetting their edges so that the wire will remain firmly in place.

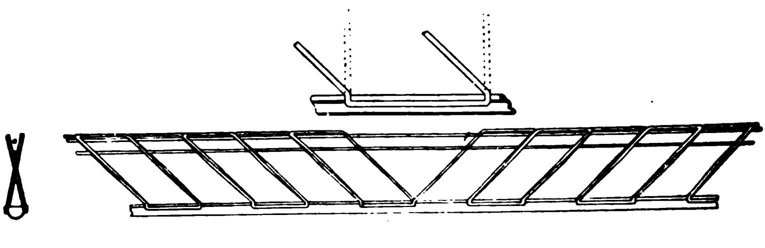


Fig. 98B—Maxwell Trussed Bar.

The arrangement of the web metal is shown in Fig. 98B.

The inclined position of the web metal reinforces the beam against shear and diagonal tension and its rigid attachment provides mechanical bond. Four sizes of bars are manufactured having areas and weight as follows:

	(1)	(2)	(3)	(4)
Area	0.315 sq. in.	0.624 sq. in.	1.11 sq. in.	1.656 sq. in.
Weight ...	1.07 lbs.	2.12 lbs.	3.77 lbs.	5.62 lbs.

The patents for this system are controlled by the American Concrete Steel Company, Detroit, Mich.

The Pin Connected Girder Frame.—This system consists of a unit frame formed of rigidly connected straight and bent bars and stirrups. The bent bars are formed into loops at the ends of the girder and are bent back parallel with the axis of the girder to form an upper reinforcement at its ends. Two systems of girder are joined together over the supports by links and pins thereby securing a rigidly connected top reinforcement in the region of negative bending moment over the

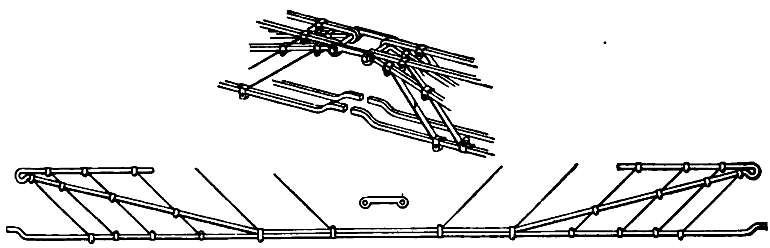


Fig. 98C.—Pin Connected Girder Frame.

supports. The bars and stirrups are fastened together by a collar and wedge device. A number of the collars or clamps are turned down to support the reinforcement at the proper elevation in the forms. The arrangement of the metal and method of connection will be understood from Fig. 98C.

These girder frames are fabricated complete at the shops before shipment. The operation of placing the reinforcement consists simply of dropping the frames into the forms and connecting them by means of the links and pins. The concreting is then placed. The General Fireproofing Company, Youngstown, Ohio, manufactures and markets this girder frame.

CHAPTER XIV.

STYLES OF SLAB REINFORCEMENT.

If some one of the various arrangements of reinforcements described above be placed at uniform intervals and parallel to each other in a concrete mass of moderate thickness, the combination forms a reinforced concrete slab. When transverse rods are used in connection with the carrying reinforcements, a latticed reinforcement results. When the transverse rods are omitted, the system is called an independent bar reinforcement. Lattice reinforcements are particularly well adapted to the construction of floor slabs. Even when independent bar reinforcements are used it is considered good practice to use small transverse rods, spaced from 2 to 3 ft. apart, to prevent shrinkage cracks.

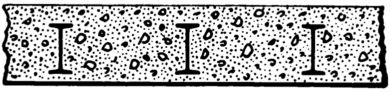


Fig. 99.—Slab with I-Beam Reinforcement.

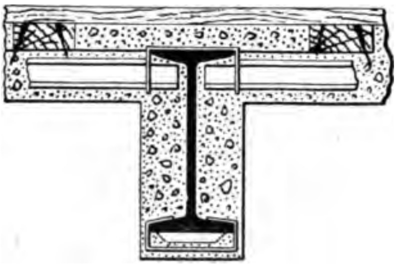


Fig. 100.—Slab with Columbian Bar Reinforcement.

Independent Bar Reinforcement.—Probably the earliest and simplest form of this class of reinforcement is that of a series of small I-beams imbedded in and supporting the concrete, which fills the space between them, as shown by Fig. 99. By retaining the same thickness of concrete, reducing the size of the beams and placing them near the lower face of the concrete, the construction approaches a true reinforced concrete system. Any one of the large variety of merchantable iron or steel shapes may replace the I-beam

A specially rolled shape is used in the Columbian fireproof floor. Figure 100 shows the Columbian bar supported at the ends by straps hung from the top flange of the beams. Two types of

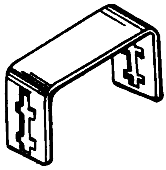


Fig. 101.—Hanger for Columbian Bars.

bars are used. The general shape is shown by the hole in the hanger, shown by Fig. 101, and the form of section, shown in Fig. 102. The smaller bars are 1, 2 and 2½ ins. deep, and are used in spans of from 5 to 11 ft. and a thickness of slab of 3-3½ and 4 ins. The second form are 3½, 4¼, 5 and 6 ins. deep and

Fig. 102.—Typical Cross-Section of Columbian Bar.

are used in spans from 12 to 20 ft., with a thickness of slab of 5 5¾-6½ and 7½ ins., respectively.

Monier Reinforcement.—The earliest and simplest form of latticed reinforcement is the Monier trellis (Fig. 103). This

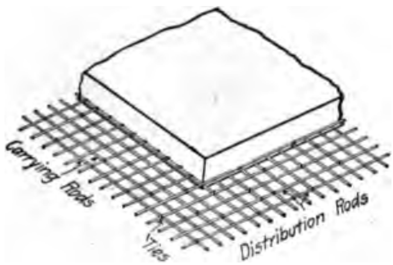


Fig. 103.—Slab with Monier Trellis Reinforcement.

consists of two series of parallel round rods crossing each other at right angles. The lower rods, called the carrying, or resistance rods, are placed in the direction of the span of the slab and form the resisting elements. The upper rods, called distribution rods, perform the two-fold function of holding the resistance rods at proper intervals apart and of distributing the load to them. Where the slab rests upon rectangular supports, the carrying rods take the direction of the shortest span. When the slab is supported on all four sides the distribution rods add their strength to the carrying rods. They are usually of smaller diameter than the carrying rods.

In the Monier system the rods are tied together at occasional

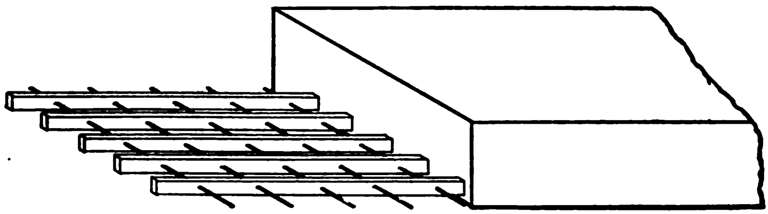


Fig. 104.—Slab with Hyatt Bar Reinforcement.

points by wrapping the intersections with wire. When the two series of rods are firmly bound together, the distribution rods prevent the carrying rods from slipping in the concrete when highly stressed. There are a number of methods of making a rigid connection between the two systems.

Hyatt System.—We have already mentioned the Hyatt system, in which the distribution bars are small, round rods passing

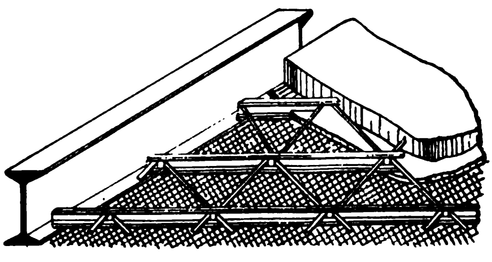


Fig. 105.—Slab with Donath Reinforcement.

through holes pierced in flats, placed on edge. These latter constitute the carrying rods (Fig. 104).

The Donath System.—This is another modification of the Monier system, using iron flats placed on edge and sometimes single or double T-shapes for the carrying bars, as shown in Fig. 105. The distribution bars are iron flats, and the two systems are bound firmly together at the points of contact. In this system pieces of sheet iron, bent in the form of an S, are sometimes used for the reinforcement.

Figure 106 shows an example of the Muller system, in which both series of flats are placed on edge.

Expanded Metal.—Expanded metal, invented by J. F. Golding, is a form of reinforcement in which a perfect union exists be-

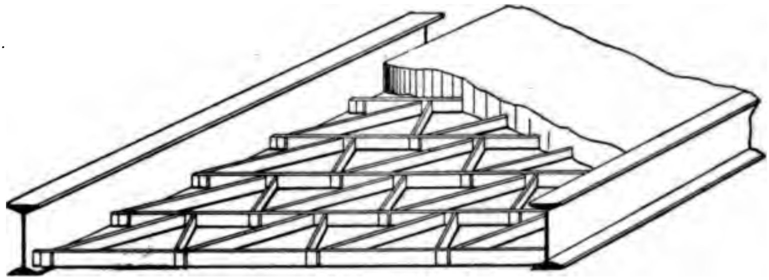


Fig. 106.—Slab with Muller Reinforcement.

tween the two systems of rods. It is a mesh work formed from a sheet of soft steel by slitting and opening or expanding the metal with meshes in a direction normal to the axis of the sheet. (Fig. 107.)

It is necessary to use a soft steel of very high quality in its manufacture, as only the highest quality of metal will stand, with-

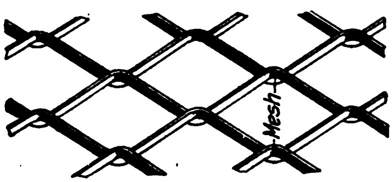


Fig. 107.—Expanded Metal.

out rupture, the process of expanding, which stretches the sheet to a width of from three to eight times its original dimensions. This material is manufactured in sheets 8 ft. long and from 12 to 72 ins. wide. Steel from No. 27 gauge, with 3/8 in. meshes up to No. 3 steel, with meshes 5 ins. by 12 ins., is used in the manufacture of expanded metal.

The mesh which is most frequently used in reinforced concrete work is the 3 in. mesh, No. 10 gauge steel. The cost of this steel is from 3 to 5 cts.per sq.ft. Expanded metal is used for slab reinforcements up to 8 ft. in the same manner as the Monier mesh. It is used in a large variety of ways, some of which will be illustrated in succeeding chapters. In the use of expanded

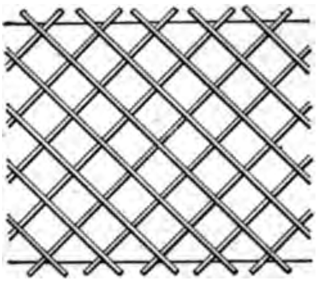


Fig. 108.—Schluter Trellis Reinforcement.

metal, care must be taken to place the longitudinal axis of the sheet parallel to the directions of the principal stress.

Schluter System.—Various modifications of the Monier system are obtained by changing the form of the mesh and the arrangement and size of the rods. In the Schlüter system (Fig. 108) two series of rods of the same diameter form an angle with the

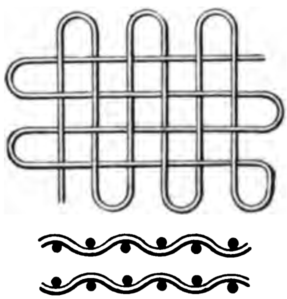
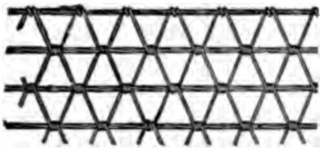


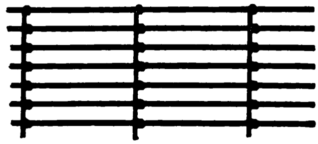
Fig. 109.—Cottancin Reinforcement.

direction of principal flexure. These two series are of equal value and, as in the Monier system, are joined at occasional intersecting points. By reducing the size of the rods to $^3/_{16}$ in. or less, they become flexible and a system of wire reinforement is obtained. Several kinds of meshing are in use. By interlacing the wires, they may be woven into a metal webbing.

American Steel & Wire Co.'s Fabric.—A variety of wire mesh reinforcement is manufactured by the American Steel & Wire Co., of Chicago, Ill. Two kinds of mesh, triangular and rectangular, are used, having longitudinal strands of ¼ in., Nos. 4, 5, 6, 8, 10, 12 and 12½ wires. The cross wires are Nos. 12, 12½ or 14 gage. For the cross wires 2 and 4-in. mesh are used for the triangular reinforcement and 6 and 12-in. for the rectangular reinforcement. In the triangular reinforcement, one, two and three wires are used for the



4-in. Triangular Mesh, Stranded Longitudinals.



Square Mesh, Stranded Longitudinals.

Fig. 109A.

longitudinal strands; when two or three wires are used the wires are twisted together. The method of connecting cross and longitudinal wires is shown in Fig. 109A.

Kahn Rib Metal.—This metal is manufactured by the Trussed Concrete Steel Co., Detroit, Mich., and consists of nine longitudinal ribs rigidly connected by light cross members as shown in Fig. 109B. The fabric is manufactured from specially rolled plates having a surface of half round ribs connected by thin flat webs. A portion of the web metal is

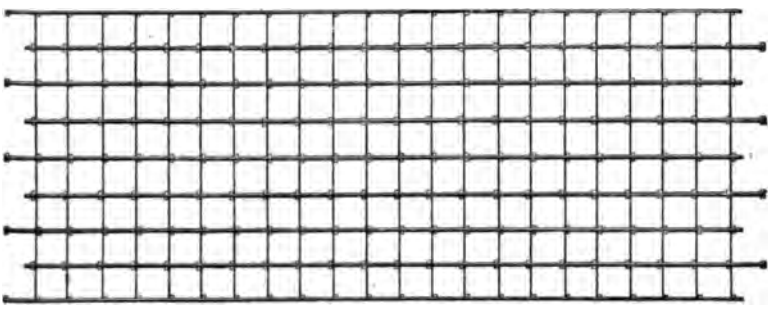


Fig. 109B—Kahn Rib Metal.

punched out and the mesh formed by expanding the sheet by pulling the ribs out sidewise until the connecting cross members are in a position at right angles to the main ribs. The standard sheets are manufactured with meshes of from 2 to 8 inches and in lengths of 12, 14 and 16 feet. The properties of the Kahn Rib Metal are given in the following table:

TABLE OF PROPERTIES OF KAHN RIB METAL.

Size No.	Distance Center to Center of Bars.	Average Sectional Area in Sq. In. per Ft. in Width.	Size of Standard Sheets	Width of Slab Reinforced by Standard Sheet	Safe Tensile Stress per ft. of Width of Slab.	Sq. Ft. per Lineal Ft. of Standard Sheet
2	2 in.	.48	17″ x 12, 14 or 16′	18 in.	7680	1.42
3	3 in.	.32	25″ x 12, 14 or 16′	27 in.	5120	2.08
4	4 in.	.24	33″ x 12, 14 or 16′	36 in.	3840	2.75
5	5 in.	.19	41″ x 12, 14 or 16′	45 in.	3040	3.42
6	6 in.	.16	49″ x 12, 14 or 16′	54 in.	2560	4.08
7	7 in.	.14	57″ x 12, 14 or 16′	63 in.	2240	4.75
8	8 in.	.12	65″ x 12, 14 or 16′	72 in.	1920	5.42

Staple-Locked Fabric.—A rectangular mesh fabric consisting of steel wire of high tensile strength locked together by a staple-lock, as shown in Fig. 109C, is used for floors and roofs, with plain round rods for beam and girder reinforcement, by the American System of Reinforcing, of Chicago, Ill. The

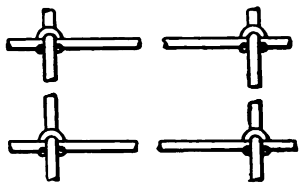


Fig. 109C—Staple Locked Fabric.

standard meshes consist of 5 Nos., 7 and 9 carrying wires, all having No. 11 cross wires, made to form 6 x 6-in., 4 x 12-in., or 4 x 6-in. meshes. The fabric comes in standard widths of 3, 4 and 5 ft., 200 lineal feet in a roll. On special order any size mesh with any size wire may be obtained.

Cottancin System.—Figure 109 illustrates the Cottancin system of reinforcement.

Lock Woven Wire Fabric.—This fabric is woven from high carbon steel wires, which cross each other at right angles and are locked at the intersection by means of No. 9 wire, twisted around the strands, as shown in Fig. 110. This fabric is made from 56 to 88 ins. wide and is put up in rolls of from 330 to 500 lin. ft. The longitudinal wires are usually spaced 4 ins. on centers, and

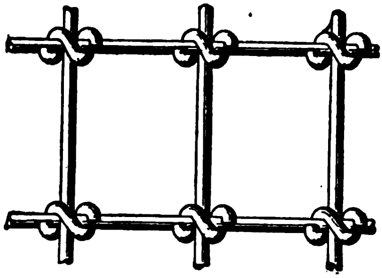


Fig. 110.—Lock Woven Wire Fabric.

the transverse wires 6 ins. The fabric may be woven of any gauge wire desired, and with various meshes.

Tie-Locked Fabric.—This fabric is manufactured by the International Fence & Fireproofing Co., of Columbus, Ohio. The above named company uses a tie lock for uniting the wires of their fabric. Fig. 111 shows the method of locking the wires. The fabric (Fig. 112) is made in widths of 4, 5 and 6 feet and in

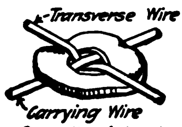


Fig. 111.—Tie-Lock.

lengths of 200 feet or as ordered. Wire in gauges from No. 18 to No. 6 is generally used, although it may be had up to $\frac{1}{4}$ to $^5/_{16}$ and $\frac{3}{8}$ in. for the longitudinal wires. The sizes of mesh vary from 4 ins. up for the spacing of longitudinal wires. The transverse wires are invariably spaced 6 ins. apart. This fabric is galvanized and is on that account especially adapted for use with cinder concrete

Table LXI. gives size, weight and strength of wire for tie-locked wire fabric.

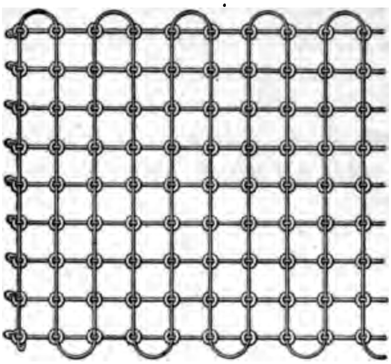


Fig. 112.—Tie-Locked Wire Fabric.

TABLE LXI.—GIVING SIZE, WEIGHT AND STRENGTH OF WIRE FOR TIE-LOCKED WIRE FABRIC.

Number by wire gauge.	Weight per ft. lbs.	Tensile strength of wire in lbs.	Diam., ins.
6	.09566	2,475	0.190
7	.08115	2,136	0.175
8	.06786	1,813	0.160
9	.05571	1,507	0.145
10	.04477	1,233	0.130
11	.03658	1,010	0.1175
12	.02922	810	0.105
13	.02268	631	0.0925
14	.01697	474	0.080
15	.01299	372	0.070
16	.00985	292	0.061
17	.00729	222	0.0525
18	.00539	199	0.045

Welded Wire Fabric.—The Clinton Wire Cloth Co., of Clinton, Mass., manufacture a fabric in which the wires are welded to-

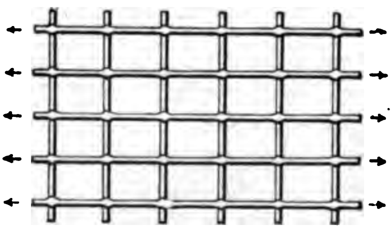


Fig. 113.—Clinton Welded Wire Fabric.

gether at intersecting points. Fig. 113 shows the general character of this fabric. The mesh may be had from 1 in. up, and of any desired size wire. From a theoretical standpoint this fabric should be an ideal reinforcement for slab construction, as the size of the wire and the spacing may be varied to give the

Fig. 114.—Slab with Suspended Wire Mesh Reinforcement.

necessary area of metal for any given load and span. The cross-wires being rigidly fixed to the carrying wires the latter are positively prevented from slipping in the concrete This fabric may also be obtained in long rolls.

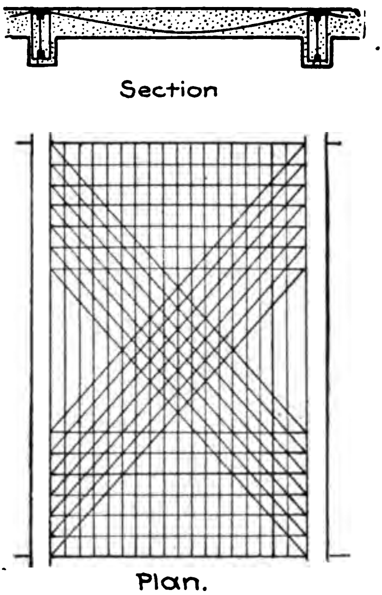


Fig. 115.—Matral System.

Manner of Using Wire Mesh.—These various systems of flexible wire meshings may be applied to thin slabs in the same manner as the Monier trellis, but it is hardly rational to consider that they act in the same manner as the larger rods in slabs where considerable resistance is needed. They are most frequently used, as is sometimes the Monier rod trellis, suspended in a curved form, after the type of reinforcement shown in Fig. 114, from beam to beam or girder to girder and act as a suspension support to sustain the concrete filling. They are often used in floor panels of considerable span, strengthened by heavier wire cables or large steel rods woven through the meshing.

Matrai System.—The Matrai system (Fig. 158) may be taken as an example of a number of systems developed from the type shown in Fig. 115. The metal network is formed of wires suspended from fixed points and allowed to assume the form of catenary curves. These wires may cross diagonally as well as in series parallel to the sides of the supporting framework, and are buried in a concrete slab. The wires alone are the carrying elements, the concrete acting only as a distributing and protecting medium.

CHAPTER XV.

STYLES OF BEAM REINFORCEMENT.

When the span passes a certain limit or the loads are very heavy the plain slab is not an economic form of construction, owing to the large quantities of concrete needed. By concentrating the metal within narrow limits and increasing the depth of the concrete, reinforced beams are obtained. Beams may be used in connection with slabs, the latter acting as a top flange to them and span-

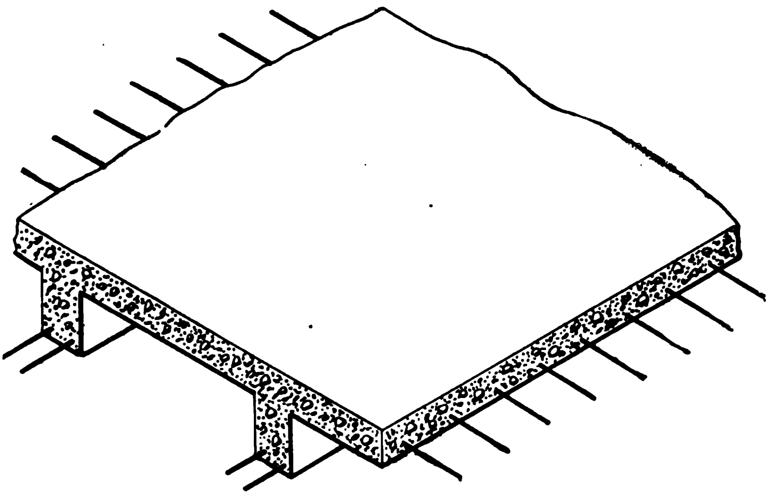


Fig. 116.—Typical Ribbed Slab Construction.

ning between them. This gives us the ribbed slab. When the beam is considered as acting independently of the slab as far as its function as a carrying member is concerned, we obtain the ordinary reinforced beam or girder. Figure 116 shows the ordinary type of ribbed slab.

For the purpose of calculation, the rib with the adjacent portions of the slab are considered as acting together and the combination is called a Tee-shaped beam. The portion above the neutral axis of the system acts as the upper flange, and is in compression. Usually reliance is placed on the reinforcement alone to care for the tensile stresses, and the concrete below the neutral axis may be considered as acting as the web of the beam. A minimum amount of concrete is used for the web, only enough being used to protect the metal reinforcement and transmit the shear.

When no slab is used, or the slab is not considered as furnishing flange area to the beam, the concrete above the neutral axis of the rectangular beam must care for all compressive stresses. It is often customary to supplement this by a compression reinforcement; this leads to the use of double reinforcements.

Reinforcements Used for Beams.—The various types of reinforcements already explained may in general be used in both types of beams. However, as the shearing stresses are higher in beams than in slabs, it is necessary to give them more careful consideration. A large number of systems of vertical reinforcements have been invented, but space only permits of the description of a few of them, and these may be considered as representing the whole. In general, it is considered good practice to use a number of small bars closely spaced, in preference to one or more bars of large area. We will, therefore, omit descriptions of beams or girders using I-beams or large shapes for tensile reinforcement. It is sometimes customary to manufacture beams in advance and put them in place during construction. The rectangular rib is the usual form used for this purpose.

The simplest form of ribbed slab consists of a plain concrete slab acting as a flat arch between ribs, spaced at short intervals. The latter are reinforced by one or more plain rods near the lower face. When it is necessary to reinforce the slab, some form of reinforcement already described under "Slabs" may be used.

Some engineers think it unnecessary to make a special provision for horizontal shear other than to use some kind of deformed rod, but this is by no means a universally accepted belief. Perhaps the simplest form of reinforcement is to use alternate straight and bent bars, the latter bars being bent at or near the quarter points and rising gradually to the upper part of the beam at the ends. The addition of stirrups gives us the Hennebique system.

Boussiron uses a special form of stirrup of hoop iron, bent into a V in the longitudinal direction of the beams as shown by

Fig. 117. By increasing the number of bent bars we obtain the Locher system. This consists of a series of bars having their

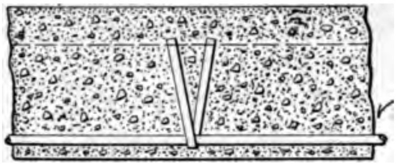


Fig. 117.—Bousseron Stirrup.

middle portion straight and curved upward at the ends, with the intention of being as near normal as possible to the direction of the maximum tensile stresses, thereby decreasing the tendency

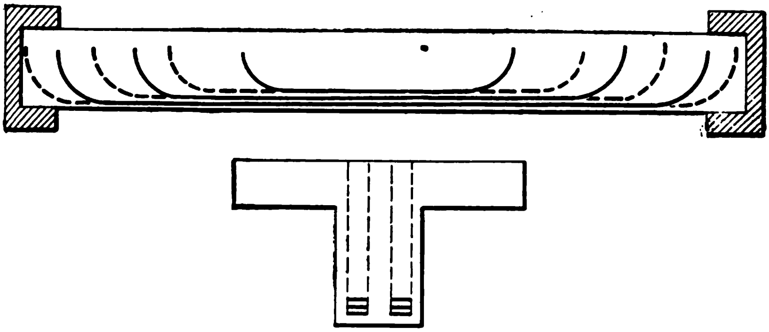


Fig. 118.— Locher Beam Reinforcement.

toward sliding or slipping along the length of the reinforcement. Either round or flat bars are used. Fig. 118 shows the disposition of the bars in the beam.

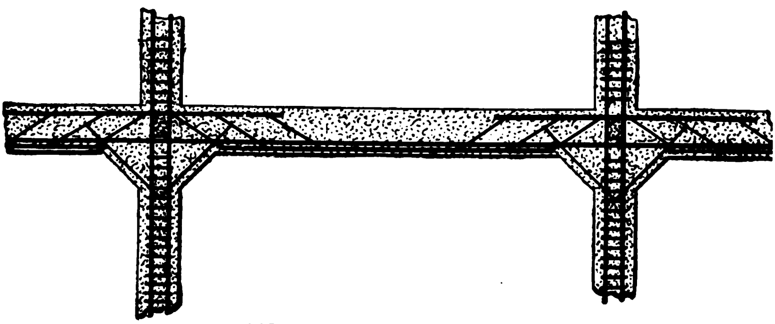


Fig. 119.—Cummings Beam Reinforcement.

A system analogous to the preceding is the invention of Robert A. Cummings, of Pittsburg, Pa. It consists of a series of rods, plain or deformed, bent into the form shown in Fig. 119. This reinforcement is especially adapted to the construction of heavy beams. Ample provision against shearing is provided and the metal is concentrated near the center of the beam where the maximum tensile stresses occur.

The Kahn bar system is particularly adapted to the construction of beams. One or more bars may be used for the bottom flange. The Kahn bar is shown in Fig. 93, while Fig. 120 shows the manner of use of this system in floor construction. When the beam is continuous over a support, inverted bars may be placed in the top flange at the supports and extending over the region of tension in accordance with the principle explained in connection with Fig. 70.

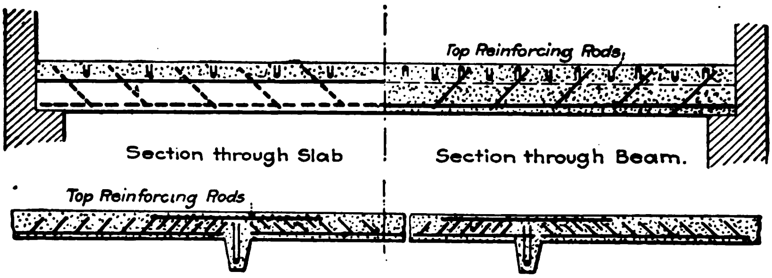


Fig. 120.—Floor Construction Using Kahn Bars.

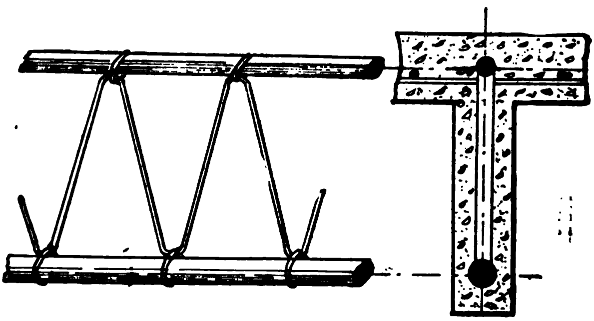


Fig. 121.—Coignet Beam Reinforcement.

The Coignet system (Fig. 121) has both upper and lower bars. These are connected by a light web formed of hoop iron fastened alternately to the upper and lower bars, thus forming a light truss.

Pavin de Lafarge has invented a system very similar to the

Coignet. In it the double reinforcements are tied together by transverse reinforcements, consisting of wires wrapped around

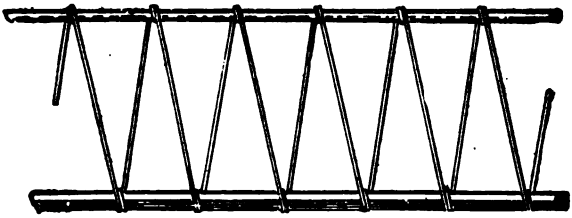


Fig. 122.—Beam Reinforcement of Pavin de Lafarge.

each rod and connecting the top and bottom rods, as shown in Fig. 122.

The section of beam shown in Fig. 123 is representative

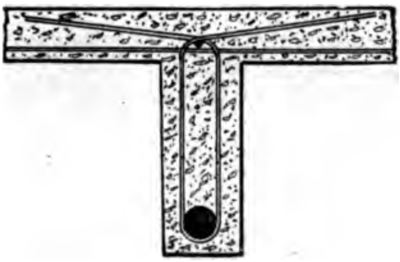


Fig. 123.

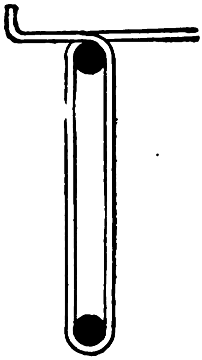


Fig. 124.

of a large number of systems using double reinforcements connected by light rods or wires; the latter often acting as a reinforcement for the floor slabs.

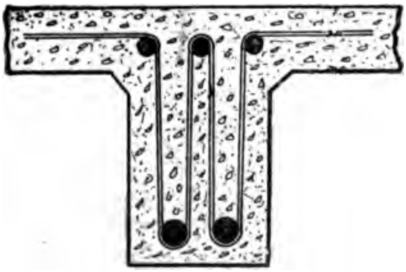


Fig. 125.

Figs. 124 and 125 show other varieties of this form of reinforcement.

The Cottancin system consists of a series of wires woven into a kind of metallic mesh. Fig. 126 shows its general character.

The Chaudy system (Fig. 124) is analogous to the Crèches

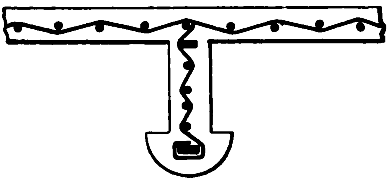


Fig. 126.—Cottancin Beam Reinforcement.

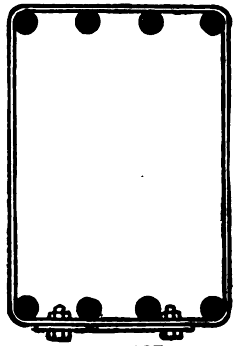


Fig. 127.

beam; the upper and lower bars being joined by round iron or hooping stirrups wrapped around them. When a number of

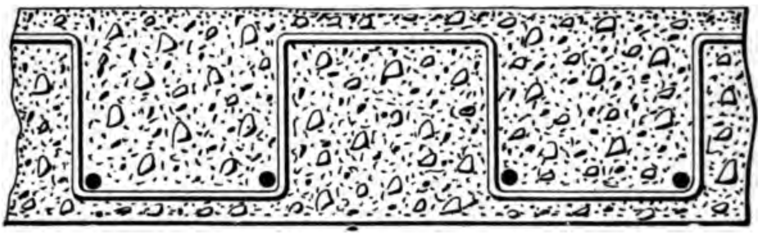


Fig. 128.—Chaudy Floor Slab Reinforcement.

bars are used, all are included within the same wrapping wire. M. Chaudy in reinforcing his beams uses double reinforcement.

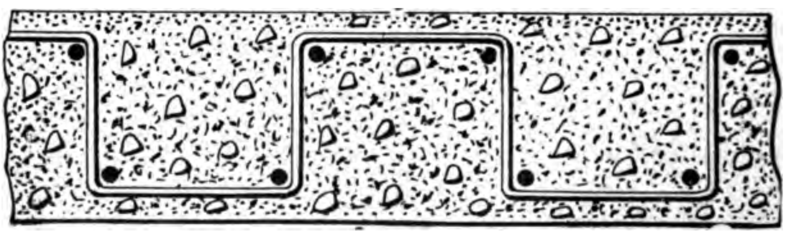


Fig. 129.—Chaudy Floor Slab Reinforcement.

He considers that the metal should be used to carry both the tension and compression, using the concrete only for enclosing it and to care for compressive stresses due to shearing. His opinion is that there should be a rigid connection between the upper and lower reinforcements. Chaudy also uses flanges composed of angles connected by either vertical or diagonal flats. These flats are connected to the angles by small rivets and are bent down upon the back of the angles, as shown in Fig. 91. In the construction of floors he used the tooth or rack system, as shown in Figs. 128 and 129.

Degon replaces the hoop iron stirrups with heavy wire ties, as he considers the former do not have the proper contact with the main bars, and by the use of light rods or heavy wires, he is enabled to tie the two materials together more perfectly. For beams he uses a double type of reinforcement, the upper rods being $^7/_{10}$ of the sectional area of the bottom ones. The bottom

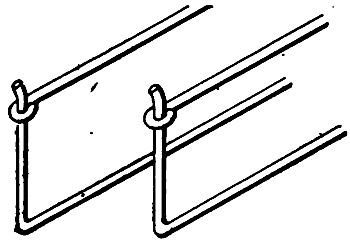


Fig. 130.

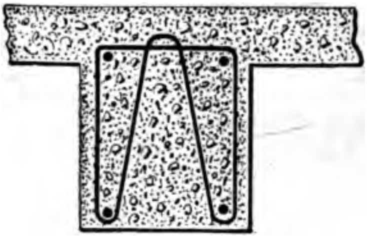


Fig. 131.

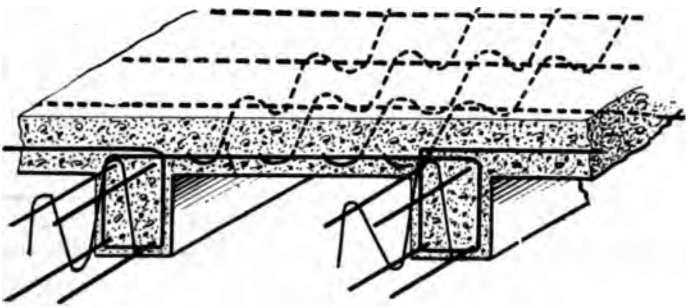


Fig. 132.

rods are bent upward at the ends so the top rods may be hooked around them, as shown in Fig. 130. The transverse reinforcements are bent in several forms, one of which is shown in Fig. 131. In this system the wrapping wires pass also in a longitudinal direction along the web of the beam. In floor slabs the transverse rods pass from bar to bar in an undulating fashion, as shown by Fig. 132.

The Hennebique System.—This system of construction is the invention of Mr. François Hennebique, and has been very extensively used for beams, girders and thin floor slabs. The principal features consist of two round rods with split ends, one rod being perfectly straight and the other bent upward at a point

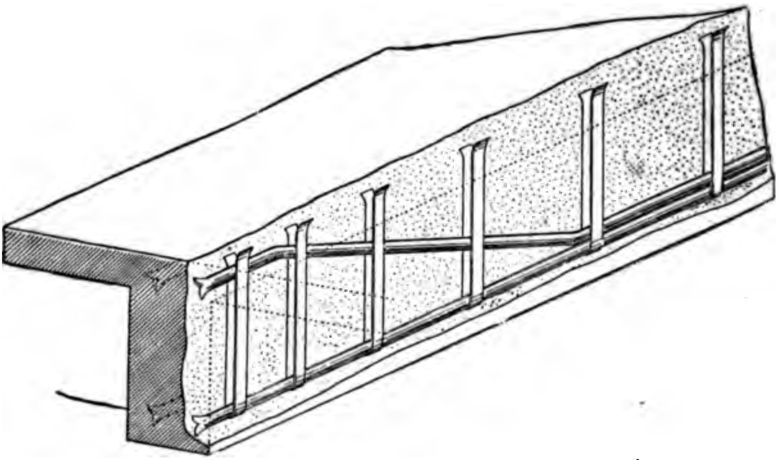


Fig. 133.—Typical Hennebique Beam Reinforcement.

about one-third of the span from the supports, for the purpose of resisting the shearing stresses at the ends. Another feature of the system is the use of hoop iron stirrups at intervals to strengthen the beam against horizontal shear. Both bars are included within the same stirrup, but in some forms of construction bent and straight bars are used alternately. Figure 133 shows

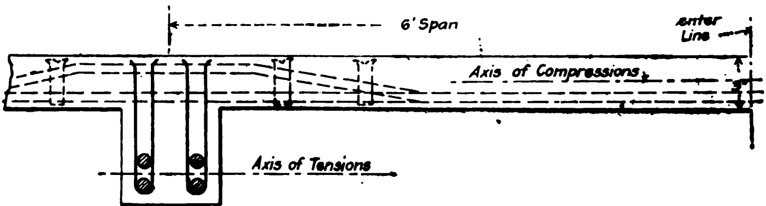


Fig. 134.—Hennebique Slab Reinforcement.

the ordinary type of Hennebique beam as applied to floor construction. The slabs may or may not be reinforced. Figure 134 shows a cross section of a floor having the slab reinforced.

In important construction reinforcement is added to the top flanges. Stirrups are placed astride of these rods and extend downward into the concrete. Round bars are used exclusively in this system. A special arrangement of the metal with transverse rods to tie the whole together and avoid longitudinal cracks is used for beams of long span.

Coularou System.—The Coularou system (Fig. 135) resembles the Hennebique. The stirrups, however, are inclined at an

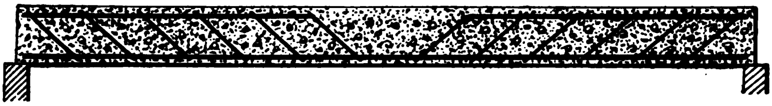


Fig. 135.—Coubarou Beam Reinforcement.

angle of 45°, and their spacing increases from the supports toward the middle of the span. Each stirrup consists of a plain round rod hooked around the upper and lower reinforcement. The upper reinforcement is of light section and is parallel to the

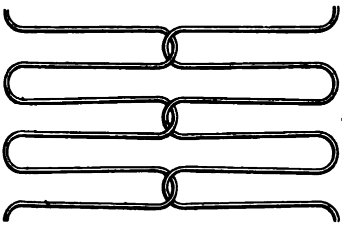


Fig. 136.

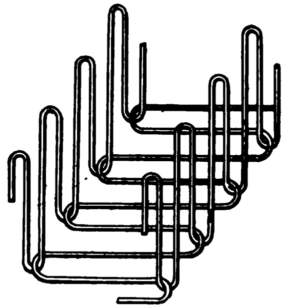


Fig. 137.

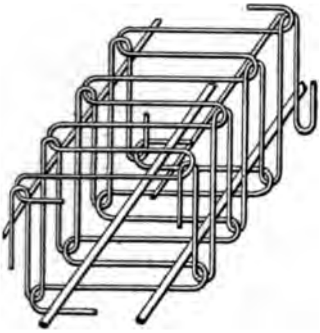


Fig. 138.

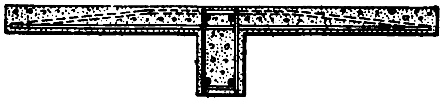


Fig. 139.

Figs. 136-139.—Maciachini Reinforcement for Beams.

lower bar, where the stirrups are necessary. Near the middle of the beam the upper reinforcement is bent downward at an angle of 45° and joins the lower bar and is parallel to it over the central portion of the beam.

Maciachini System.—The object of this system is to obtain for beams the advantage gained in the construction of columns by hooping the same. The hooping of beams is an extremely difficult operation. S. Maciachini undertakes to obtain the hooping effect in the following manner: Hooping wires of a suitable diameter and as long as possible are bent up and down before being placed in position, the height being that of the width or depth of the beam less about 1¾ ins. to allow for a covering of about ⅞ in. of concrete on all sides. The bottom and side hoopings are placed together, as shown in Fig. 136, so that when all connected these reinforcements appear as shown in Fig. 137. After the ⅞ in. of concrete has been deposited, this meshing, together with the bottom rods at the angles, is put in place and

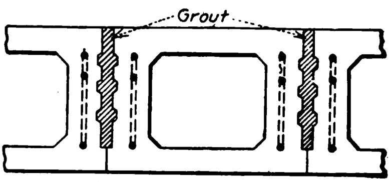


Fig. 140.—Siegwart Hollow Beam.

the filling is brought up and well rammed until it reaches the level of the top rods. These are then put in place and the top portion of the hooping is threaded through the top loops of the sides and bent backward and forward, as shown in Fig. 138. After this operation is completed the remainder of the concrete is added. Fig. 139 shows a cross section of a beam of this form.

Lattice Trusses.—Lattice trusses are also used for reinforcing concrete girders. Matrai makes use of this form to support his girders. He also sometimes uses rolled beams. He greatly reduces the bending moment in his beams and girders by

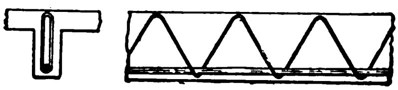


Fig. 141.—De Valliere Beam Reinforcement.

attaching the ends of the wires and cables supporting the floors as near as possible to the end of the beam. See the arrangement of wires, as shown in Fig. 158.

Siegwart System.—This system consists of a hollow beam reinforced by round iron rods, its top face forming the floor slab and its bottom face the ceiling. The beams are moulded in sections about 10 ins. wide and are constructed in advance. The open spaces between the beams are filled with cement grout and the whole mass becomes a more or less perfect monolith. These floors cost between 15 and 20 cts. per sq. ft., according to the span and load. Fig. 140 shows the nature of this hollow reinforced beam.

De Valliere System.—In its simplest form, this system consists of a main reinforcing bar with a web reinforcement of heavy wire bent in the form shown in Fig. 141. This system resembles that of Pavin de Lafarge, but in this case the top rod with its loop is omitted.

The Visintini System.—This system is the invention of Mr. Franz Visintini, of Zürich. The Concrete Steel Engineering Co., of New York, controls the patents for its use in America. This system is used in the construction of floors and roofs, and consists

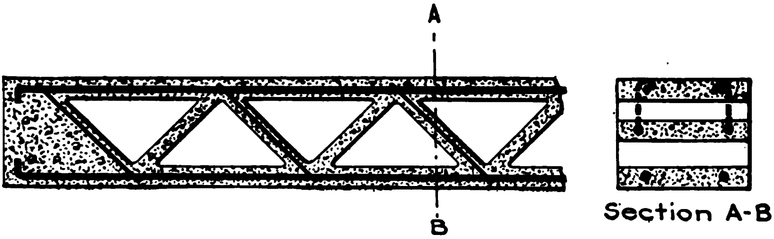


Fig. 142.—Visintini Beam.

of shallow beams moulded in advance. Floors are made up of a series of these beams placed side by side. The beams are usually moulded in widths of from 6 to 12 ins., and are 6 or 8 ins. in depth. The beams are in reality shallow Warren trusses, and are reinforced as shown in Fig. 142. No reinforcement is used in the web members, which are strained in compression. For deep girders spanning between columns, a similar beam, usually trussed according to the Pratt system, is used. In this case the verticals are in compression and are not reinforced. Round rods are used for the main reinforcements, and sometimes flats are used for reinforcing the diagonals in the shallow floor beams. The advantages claimed for this system are economy in material, careful inspection and first-class workmanship during construction, and a definite action of stresses, according to the system of trussing used.

CHAPTER XVI.

CURVED PIECES STRAINED IN FLEXURE.

If a curved piece, resting upon two supports, be acted upon by external forces, the resulting deformation produces either a thrust or pull at the supports, and the internal stresses developed by the action of these forces may be either compression or tension, but usually, on account of the form of the piece, or because of unequal distribution of the load, flexure also exists. If the piece is loaded on the convex side, it is in compression and flexure. Curved pieces loaded on the convex side are used in many forms of construction, and may all be included under the general classification of arches. If the loading is on the concave side, the piece is in tension and flexure.

Arches.—These, like straight pieces, will be divided into two classes: Those of uniform thickness in the direction parallel to the axis, and ribbed arches. In the ribbed form, the ribs are sometimes constructed independently in the form of arches of rectangular section.

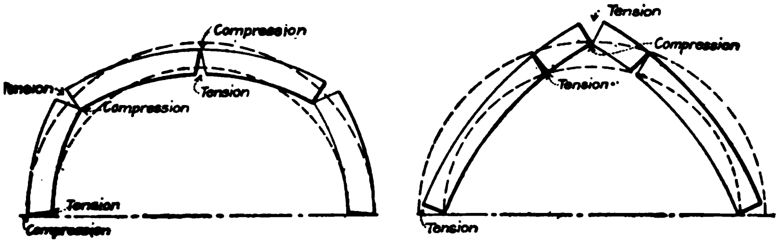
Stresses in Arches.—The arch acts under compression and flexure. On account of the flexure both tension and longitudinal shear exist. In this form of construction the object of the reinforcement is to supply the resistance needed by the concrete when it is subject to tensile stresses and to supplement its compressive resistance whenever it is necessary to do so. These two kinds of stresses are the most important and may be called principal stresses. Secondary stresses also exist, the most important of which is shearing stress. Provision is often also made for this stress.

Before showing the arrangement of the reinforcement, we will review briefly the usual manner of failure of arches and the location of the dangerous stresses producing such failure.

Let us consider that the arch under discussion is of concrete and fixed at the ends. The arch may fail (1) by crushing the concrete, (2) by shearing, and (3) by rotation.

(1). If the thrust at any point exceeds the compressive strength of the concrete, the arch will fail. By introducing a proper amount of reinforcing metal the compressive stress in the concrete may be kept within safe working limits. Hence, by the use of reinforcements the thickness of the concrete may be greatly reduced.

(2). As in beams, when the arch is heavily loaded, particularly if it be a flat arch, the shearing stresses at or near the ends may become dangerously high. The arch ring is generally



Figs. 143-144.—Sketches Showing Methods of Failure of Arches.

strengthened by increasing the thickness near the springing points. Reinforcement, when used, will give it additional strength.

(3). Failure by rotation. The methods of failure by rotation are shown in Figs. 143 and 144.

A flat arch fails by sinking at the crown and rising at the haunches, a deep arch by rising at the crown and sinking at the haunches. In Fig. 143, tension exists in the intrados at the crown and springing points; in the extrados at the haunches.

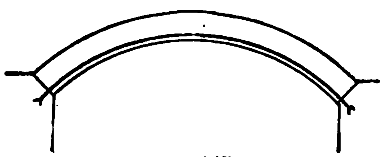


Fig. 145.

In Fig. 144 tension exists in the extrados at the crown and springing points, and in the intrados at the haunches. By examining these figures the proper distribution of the reinforcement is easily understood.

The simplest form of arch fixed at the supports is a curved ring included within two curved surfaces. As in the beam the simplest form of reinforcement is a bar placed near the intrados as shown in Fig. 145. This form of reinforcement will be insufficient if the stresses due to compound flexure are of any magnitude.

In a full centered or elliptical arch, tensile stresses occur, as shown in Fig. 143, at the haunches. Hence it is customary to strengthen the extrados by a supplementary reinforcement. In an excessively elliptical arch the joint of rupture generally occurs at a point slightly above the springing line. Hence the supplementary reinforcement begins at the springing line and extends over the dangerous space, as shown in Fig. 146. As this danger space is sometimes difficult to determine, it is often customary to extend the reinforcement at the extrados over the whole arch (Fig. 147), thereby obtaining the double reinforcement. The

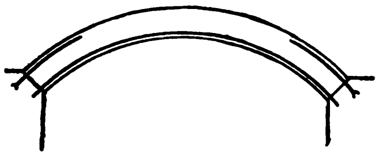


Fig. 146.

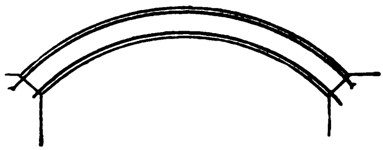


Fig. 147.

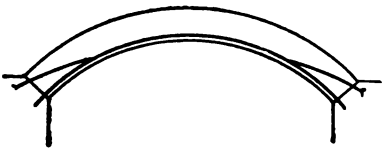


Fig. 148.

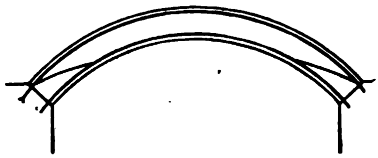


Fig. 149.

double reinforcement in addition to caring for any dangerous tensile stresses supplies additional resistance to compressive stresses.

Figure 148 is a modification of Fig. 146, the reinforcement at the extrados being bent downward to meet the reinforcement of the intrados. This form may be used in connection with a double reinforcement and we then obtain the form shown in Fig. 149. The above five forms apply to arches of constant thickness, but it is customary to vary the thickness with the pressure and bending moment. This gives a greater thickness at the springing lines than at the crown. The thickness may be increased until the extrados becomes a horizontal plane. The above systems of reinforcement also apply to this form and we obtain the distribution of reinforcements shown in Figs. 150 to 154.

Secondary Stresses.—In arches the principal stresses are generally of much greater magnitude than the secondary stresses induced by simple flexure, hence it is not necessary to make any special provision for shearing stresses. There are, however, some arch systems which use special bonding similar to that used in straight beams. Another kind of secondary stress should be considerad. When arch rings or straight pieces are subject to heavy compression stresses, a compression of the concrete takes place, and at the same time a lateral expansion. Some method of transverse bonding is necessary to take care of this secondary deformation. If the piece be reinforced against this

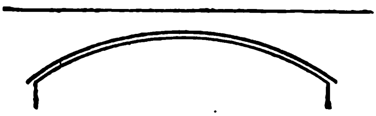


Fig. 150.

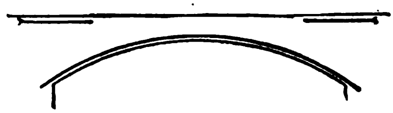


Fig. 151.

Fig. 152.

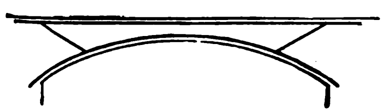


Fig. 153.

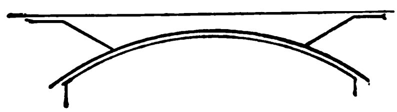


Fig. 154.

lateral expansion, its resistance to compression is greatly increased.

Systems of Reinforcement.—The systems of reinforcement described in connection with slabs may be applied to the construction of arches of uniform thickness.

The first and simplest form of arch reinforcement (Fig. 145) was first used in connection with a Monier netting. This did not materially strengthen the arch, as no provision was made to care for the tensile stresses produced by the bending moments at the extrados.

Other methods of strengthening the arch ring were devised and the reinforcement took the form shown in Figs. 146 and 148, and finally the double netting was used, as shown in Fig. 147.

In the Monier type the resistance bars were bent to the curve of the directrix of the arch and the distributing bars were straight and parallel to the axis, the two systems being bound together at intersecting points.

Expanded metal has been extensively used to reinforce arches of short span, particularly for floor systems.

The Melan system has been widely used in the construction of bridges. While this system is essentially a double reinforcement type, the reinforcements occur in a variety of forms. Steel I-beams have been extensively used, also various forms of lattice girders. The vertical bonding is sometimes omitted, and independent bars of various forms are used. These are placed parallel to the curves of the extrados and intrados.

Mr. Edwin Thacher used the reinforcement shown in Fig. 78 for a large number of his earlier bridges. The reinforcing bars are spaced at convenient intervals, and may or may not be con-

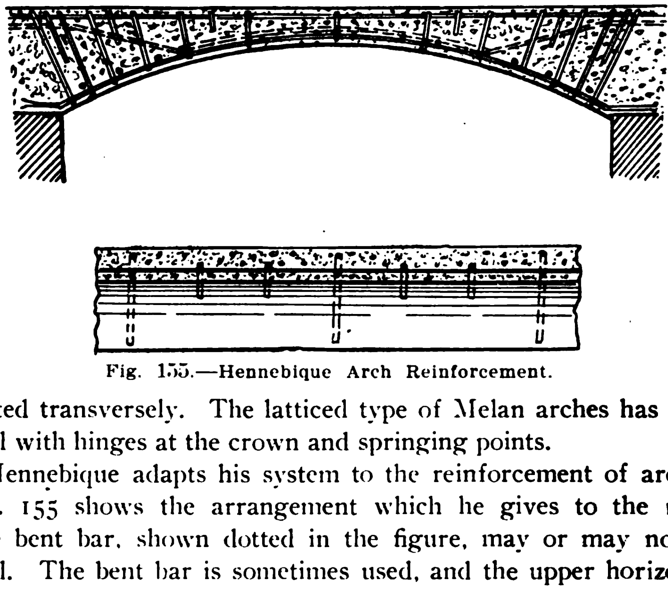


Fig. 155.—Hennebique Arch Reinforcement.

nected transversely. The latticed type of Melan arches has been used with hinges at the crown and springing points.

Hennebique adapts his system to the reinforcement of arches. Fig. 155 shows the arrangement which he gives to the rods. The bent bar, shown dotted in the figure, may or may not be used. The bent bar is sometimes used, and the upper horizontal bar omitted. As in straight girders these sets of bars are placed in the same vertical plane and are included between the same stirrups. Distribution bars may be used. These are placed parallel to the axis of the arch and above the intradosal bars and below the bent bars.

Ribbed Arches.—From a theoretical standpoint if the arch acts under compression alone, it ought to be of uniform thickness, as the thrust should be the same throughout its cross section. However, on account of flexure and the concentration of loads at certain definite points through the medium of spandrels, a greater depth and a corresponding greater moment of resistance is at times desirable. Hence, as in slabs the ribbed type gives the desired strength without an excess of concrete.

Ribbed arches are often more economical than arches of con-

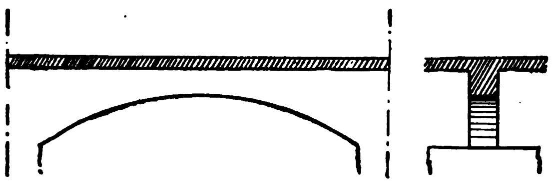


Fig. 156.—Hennebique Arch Rib and Flat Slab Construction.

stant thickness. By using a thin arch at the extrados strengthened by ribs, the amount of spandrel filling and dead load may be reduced. In this case either a curved or flat extrados may be used. The latter case gives arch ribs with a flat covering slab, as shown in Fig. 156. The Hennebique system uses the form of flat extrados. The reinforcement for the ribs is made up in the same way as in arches of uniform thickness, but in this

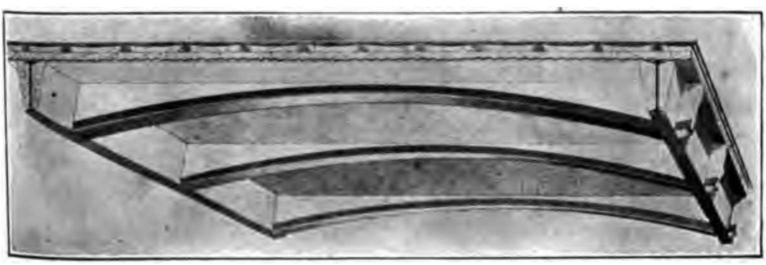


Fig. 157.—Golding Arched Floors.

case all three rods are used in the same vertical plane, viz., intradosal, bent and extradosal reinforcing bars. The slab is reinforced in the same manner as ordinary floor slabs.

In arches of large span the floor is itself sometimes arched giving an arch system, which is analogous to a ribbed floor slab with a certain amount of curvature over the whole span. The flat slab reinforced with arch ribs of moderate span has been extensively used in floor construction. The Golding floor is the most common type met with, and is shown in Fig. 157.

Inverted Arches.—A curved system of construction may be used, in which the arch is loaded on the concave side, in which

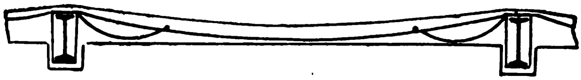


Fig. 158.—Matrai Inverted Arches.

the usual thrust at the supports becomes a pull and the piece acts under tension and flexure. M. Matrai (Fig. 158) adopts this form of construction for his system. The intrados may be either flat or curved. Fig. 154 shows the form with a flat intrados used by M. Matrai in the construction of floors.

CHAPTER XVII.

COLUMNS, WALLS AND PIPES.

Straight Pieces Strained in Compression.—In this classification belong all those structures in which compression acting in the direction of the axis of the piece is the principal stress. Secondary stresses produced by bending usually also exist. These latter stresses are due to external forces, acting normal and obliquely to the axis of the piece, or to an eccentric application of the forces producing the principal stress. Sometimes an uneven settlement of the foundation will also develop secondary stresses. Walls, columns, posts, piers, foundation piles, etc., acting in a vertical direction, come under this classification.

Disposition of the Reinforcement.—The reinforcement should be so arranged as to care for all dangerous stresses. In this class

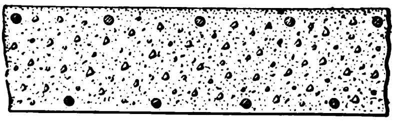


Fig. 159.—Typical Wall Reinforcement.

of structure the function of the reinforcement is twofold, viz., to take care of any tensile stresses due to bending and to supplement the compressive strength of the concrete. When used principally in the latter capacity it is often possible to reduce materially the section of the concrete. In general the reinforcement consists of straight rods placed parallel to the axis of the piece and is so arranged that the piece will resist equally well in all directions the action of the direct compression and the secondary flexural stresses.

Walls.—The reinforcements in walls, to take care of transverse bending stresses and assist in carrying the vertical loads, consist of vertical rods placed alternately near each face, as shown in Fig. 159. Reinforced walls in reality are slabs, with double reinforcements, placed on edge. Some form of transverse bonding is desirable, for when a piece is under compression, transverse distortion accompanies the longitudinal contraction due to compression. As in slabs, longitudinal rods spaced from 2 to 3 ft. apart should be placed horizontally throughout the height of the wall to prevent shrinkage cracks. Again it is

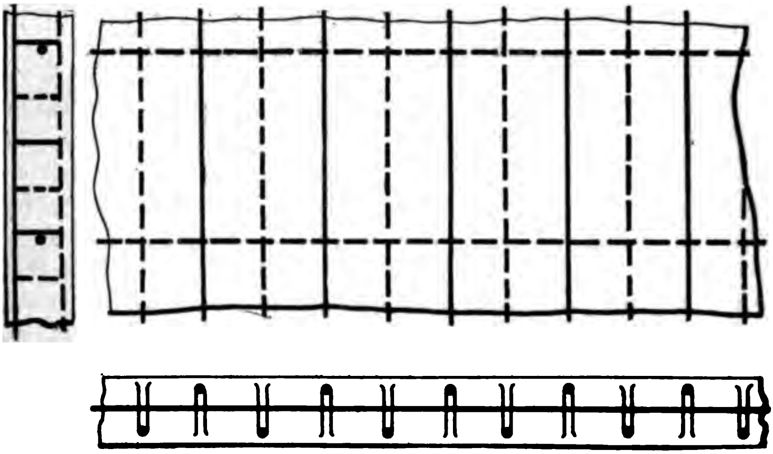


Fig. 160.—Hennebique Wall Reinforcement.

often desirable to reinforce the wall in a horizontal direction in order that it may act as a girder to distribute the loading over the foundation or to span openings; to this end, rods are placed horizontally throughout the height of the wall.

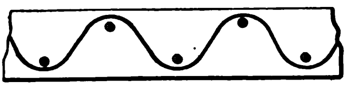


Fig. 161.—Degon Wall Reinforcement.

M. Hennebique uses vertical round rods placed alternately near each face and ties them to the opposite face by means of stirrups, as shown in Fig. 160. Horizontal rods are placed along the axis of the wall to take care of vertical flexure.

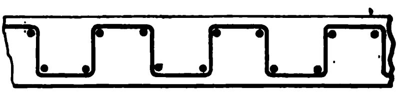


Fig. 162.—Chaudy Wall Reinforcement.

Ransome employs a similar arrangement, but uses cold twisted bars without stirrups.

The Degon (Fig. 161) and Chaudy (Fig. 162) systems for walls use an arrangement of the reinforcements similar to that employed in the construction of these floor systems.

The Monier trellis, used as in a double reinforced slab, is employed for wall reinforcement. A single trellis is often used for thin partitions. Any one of the latticed systems described under slabs may be used in a similar manner. Partitions are often made in panels moulded in advance, and put in place during the process of construction.

Columns.—Concrete and metal are used together in a number of ways in the construction of columns. Concrete was long ago used as a protecting coat against fire for all-metal columns. The column proper is usually built up of steel shapes, and surrounded with some form of metal lath or lattice, which is embedded in the surrounding concrete, as shown in Fig. 163. When used in this manner the concrete adds very little to the strength of

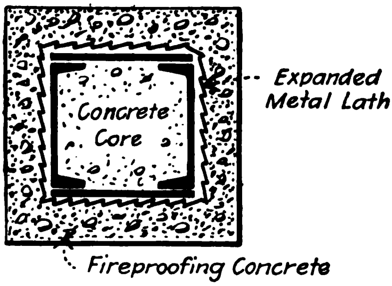


Fig. 163.—Concrete Covered Steel Column.

the column. If the interior of a hollow steel column be filled with concrete, additional stiffness is secured. Capt. John S. Sewell, Corps of Engineers, U. S. A., has used columns of this kind in building construction and states that by the use of a concrete core the saving of steel was great enough to pay for the filling. While the stiffening effect of the concrete core is somewhat uncertain, it is probably safe if large cross-sections be chosen to design the steel column in simple compression, without any allowance for flexure, dependence being placed on the concrete to furnish the necessary stiffness.

The usual method of reinforcing concrete columns is to place vertical reinforcing rods symmetrically spaced about the axis of the piece and as near as possible to the exterior faces. The columns may be square, round or of polygonal form. Usually from 4 to 20 rods, varying in diameter from 3/8 to 2½ ins., and upwards are employed. These rods are tied together at intervals of about the thickness of the column. For these ties Hennebique formerly used flat bars having holes punched in them (Fig. 164) through which the vertical rods are passed. In later construction he has used wire ties. Hoop iron straps are used in the Bousseron system (Fig. 165), while the Degon system (Fig. 166) employs wire ties bent in the form of a cross. A variety of other methods of wrapping are often employed.

The Kahn system omits the cross ties, but, owing to the peculiar form of the bar used, it is tied firmly to the mass of the concrete.

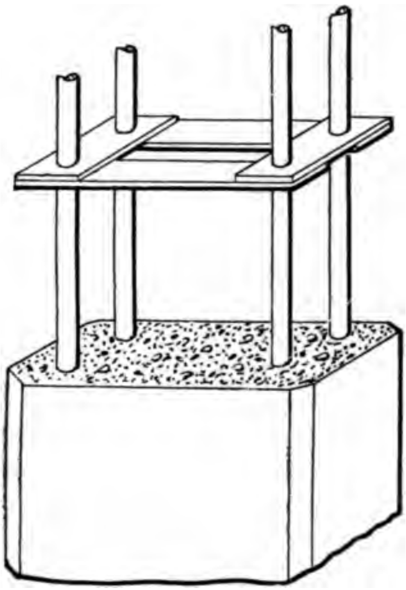


Fig. 164.—Hennebique Column Rod Ties.

Hooped Columns.—It is a well known fact that when a concrete piece is subjected to high compressive stresses, a deformation and shortening of the column occurs accompanied by a distortion of the molecular structure and swelling of the concrete in a transverse plane. M. Considère conceived the idea of increasing the compressive stress of the concrete and preventing the horizontal distortions by means of hoops or helicoidal spirals placed at or near the face of the column (Fig. 167). By this means the compressive strength of the column may be increased from 2 to 2.7 times that obtained by the usual type of reinforcement. M. Considère found that the spirals should be spaced from 1-7 to 1-10 of the diameter of the column. Rods should vary in size from ¼ to ¾ in., according to the size of the column and the load to be carried. Vertical rods are used in connection with this hooping to care for flexural stresses. The

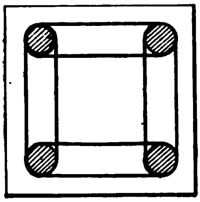


Fig. 165.—Bousseron Column Rod Ties.

confined concrete carries all the direct compression. Expanded metal has been used in place of the spiral hooping.

Piles.—Concrete piles may be reinforced by placing a single rod or shape on the axis of the piece, or they may have a number of reinforcing rods arranged as in columns.

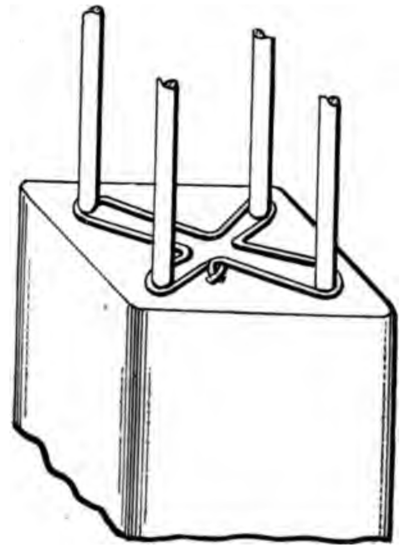


Fig. 166.—Degon Column Rod Ties.

Curved Pieces Strained in Compression.—Vaults, reservoirs, sewers, pipes, etc., under pressure, acting upon their curved sides, are included in this classification when their form and the distribution of the loading is such that the principal stress developed is compression. A parabolic arch acted upon by a vertical loading uniformly distributed over its horizontal projection will be strained in simple compression. A circular pipe submitted to a uniform normal pressure acting from without will also be in simple compression and may be considered as typical of this class of structure.

Some form of lattice reinforcement is usually employed. The resistance bars are bent or wound into circles or hoops and the distribution bars are straight and parallel to the axis of the pipe or cylinder. They are placed outside the directrix, or resistance

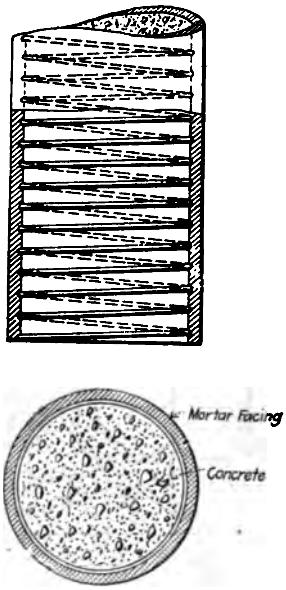


Fig. 167.—Typical Hooped Column Reinforcement.

bars. The two systems are bound together at their points of intersection by soft wire ties. The resistance bars may be in the form of hoops or manufactured of long rods spirally coiled. The rods may be reduced in size and woven into some form of wire meshing such as is used in slabs. When light wire fabric is used it is often strengthened and retained in the desired form by large rods woven spirally about it. Sometimes two or more meshes are used. Round and square rods and T, cross and I shapes are used for reinforcement. M. Bonna uses special steel-sections having the form of a Latin cross; these vary in size from 1/4 in. to 7/8 ×

1⅝ ins. M. Bordenave, another specialist on hydraulic work, uses small I-sections. These vary in depth from 5-16 in., with an area of 0.023 sq. in., to 1 in., with an area of 0.161 sq. in. This engineer uses round rods for distribution bars. These small sized shapes necessitate an extra amount of rolling during manufacture, which gives a higher elastic limit and ultimate strength than is obtained in round or square rods. The peculiar form of the rods gives a large area of contact for adhesion. The cost of manufacture is considerably higher than that of ordinary round or square bars. The reinforcement may be placed near the middle of the cement ring, but is sometimes placed near the inner

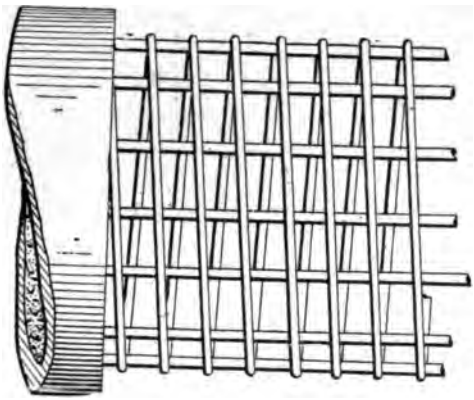


Fig. 168.—Bonna Reinforcement for Pipe.

face, as in the simplest form of Monier arch reinforcement. When a double network is used, one should be placed near the inner and the other near the outer face.

Curved Pieces Strained in Tension.—In this classification are to be found pipes and reservoirs subjected to internal pressure. The reinforcement is designed to take all the tension and is embedded within the concrete shell usually near the outer face. In this case, however, the distribution bars are placed within the resistance bars, the latter being plain hoops or spirals, as in pipes under compression. The concrete acts as a distributing and protecting medium to this skeleton work and should be able to care for all secondary stresses. All the systems which were mentioned in connection with pipes under compression may be used for pipes under internal pressure. Many of these systems have been developed in the construction of water mains and sewers.

Figure 168 shows the manner of arrangement of the reinforcing skeleton work of the Bonna system. In this system, when the pipes are small the hooping is always used in the spiral form. The longitudinal rods are notched to receive the hoop-bars at intersection points, and the joints wrapped with soft wire. When the pipes are to be used under a head of 50 ft. or more, M. Bonna places a sheet steel tube in the concrete to insure impermeability.

CHAPTER XVIII.

GENERAL PHENOMENA OF FLEXURE.

Action of Beams Under Tests.—According to the experiments of Profs. Hatt, Talbot and Turneaure, reinforced beams under test exhibited four stages of flexure.

In the first stage the combination acts as a true composite member. This period extends from the application of the load until the tensile strength at the extreme fiber of the beam, due to both

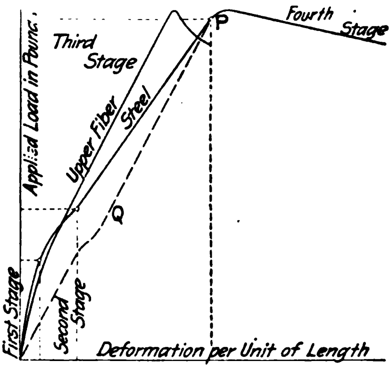


Fig. 169.—Typical Load Deformation Diagram.

its weight and the applied load, becomes equivalent to about 350 lbs. per sq. in. Under the conditions of Prof. Talbot's tests this was approximately at the ultimate strength of the concrete in tension. At the limit of this stage the deformation of the steel throughout the middle third of the beam is about 0.0001 of its length, indicating, if we disregard shrinkage stresses and use the value of the coefficient of elasticity of naked steel, that the reinforcement is stressed about 3,000 lbs. per sq. in. Up to the end of this stage the deformation curve is quite regular and the ratio between the load and the deformation is very nearly constant. Figure 169 from Prof. Talbot's discussion of reinforced concrete tests, University of Illinois Bulletin No. 1, for Sept., 1904, shows the form of deformation curve during the various stages of flexure. Throughout this stage the neutral axis is below the middle of the beam. The point on the deformation curve which shows the limit of the first stage is designated by Prof. Hatt as "Point A."

The second or readjustment stage begins when the loading exceeds the limits stated for the first stage. The steel elongates more rapidly as the loading increases, the compression of the concrete increases and the neutral axis rises. No cracks are visible to the naked eye during this period, but Prof. Turneaure found evidences of cracks when wet beams were tested by the appearance of water marks or bands. The cracks develop during the next stage. The concrete, as shown by the fine cracks not visible to the naked eye, begins to give up its tensile stress to the steel during the early part of this stage. There is a marked change in the form of the deformation curve, and its reversal of curvature near the end of this stage seem to indicate that the concrete is broken in tension through a part of the depth of the beam and that portion of the load carried by the concrete in tension has now been transferred to make additional tension in the steel. This stage lasts until the deformation at the level of the steel averages about 0.00035. The condition for the usual working load of beams is included within the limits of this stage.

During the early part of the third stage the entire tensile value of the concrete is lost and all tension is transferred to the steel. Fine vertical cracks well distributed along the middle third of the beam become visible to the naked eye and gradually grow more distinct. According to Prof. Hatt, the first crack becomes plainly visible to the naked eye under a load three and one-third times the load at "Point A," and after a deflection, which is nearly six times that at "Point A." The first crack in Prof. Turneaure's tests on 1:2:4 concrete appeared at an extension in the concrete of about 3 times that at "Point A," and in Prof. Talbot's tests on 1:3:6 concrete, at an extension of about 5.7 times that at "Point A," indicating in the former case a stress in the steel of from 12,000 to 16,000 lbs. per sq. in., and in the latter case a stress in the steel of 23,000 lbs per sq. in. These fine cracks close up when the load is removed and can not enlarge to a serious extent until the steel reaches its elastic limit. The appearance of the cracks is, however, not accompanied by any apparent change in the character of the load deformation curves, which are quite regular for both tension and compression throughout this entire period. The increments of the deformation of the steel are very nearly proportional to the increments of the load as is shown by the approximately straight line in the diagram. The compression deformation also closely approximates a straight line as shown in the diagram. The position of the neutral axis is practically constant throughout this stage. On account of the straight load deformation lines for both tension and compression, and the nearly constant position of the neutral axis, there results a definite proportion between the increment of the load and the increment of the resisting moment of the beam, if the latter be based upon observed deformation and the coefficient of naked steel. While the observed deformation may differ somewhat from the deformation necessary to give the true resisting moment of the beam during the early parts of this period, due probably to initial stresses or other causes effecting the measured deformation, it will be found that at or near the limit of this stage these deformations are almost directly proportional to the increments of the load and at the maximum load the resisting moment, calculated from the observed deformation of the steel, will, in general, be nearly equal to the bending moment of the load. Unless there be an excess of reinforcement, this stage continues until a point is reached at or near the maximum strength of the beam.

The fourth stage, or stage of failure, begins at or near the maximum load. The deflection increases if a normal amount of reinforcement is used, or, when not enough metal is used to develop the crushing strength of the concrete, i. e., not more than 1½ per cent. of metal for steel having an elastic limit of 33,000 lbs., and 1 per cent. for steel with an elastic limit of 55,000 lbs. per sq. in. The steel stretches rapidly, the cracks grow in width, the neutral axis rises, and there is a rapid increase of compression in the upper fiber of the concrete, due to the decreased compression area, until finally the concrete crushes at the top of the beam at a load less than the maximum, and after the steel has stretched considerably beyond its yield point.

According to Prof. Hatt, failure of the beam begins at a load about four times the load at point A, and at a deflection which is about nine times the deflection at point A. The steel has passed the yield point and the cracks are wide open and will not close upon the removal of the load. In Prof. Talbot's tests, the values were 4.3 and 16, and in Prof. Turneaure's tests 3.5 and 9, respectively, for ratio of load and deflection to that at point A. In Prof. Talbot's tests, soft steel with an elastic limit of about 33,000 lbs., up to 1½ per cent., was not sufficient to develop the crushing strength of the concrete. Prof. Hatt was not able to develop the compressive strength of the concrete with 2½ per cent. of steel by the application of a central load. With a high percentage of steel

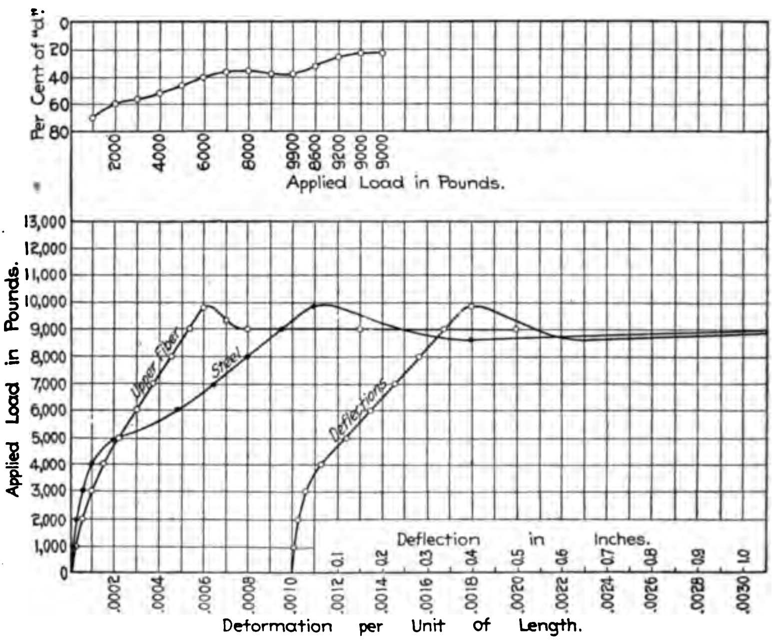


Fig. 170.—Diagram Showing Change in Neutral Axis and Deformation of Test Beam.

it is probable that the bars will slip, if not anchored, or the concrete will shear along the plane of the reinforcement before 1:2:4 concrete will fail in compression. It is evident that the range between point A and the first crack will depend largely upon the age and quality of the concrete. The poorer and dryer the concrete, the nearer will be the first crack to the point A.

The deformation curves for this, as well as for the first three stages, will be understood by an examination of Fig. 170.

It was found from the average of a number of tests that deformation at the end of the straight line for mild steel bars is about .00115, and for high steel bars .0020, which approximates closely the computed deformation for naked steel bars at the yield point, and we may conclude that the maximum load is reached at the yield point, and that the yield point may be properly taken as the ultimate strength of the beam.

It seems also true that loads which will stress the steel to its elastic limit may be calculated by using the elastic limit of naked steel for the tensile stress in the beam and neglecting the tensile stress in the concrete.

Where more than a normal amount of metal is used, the concrete at the top of the beam will fail by crushing before the elastic limit of the steel is reached. In this case the deformation curve will be somewhat different. When a deformation in the upper fibre of about .0014 is reached the deformation line for the upper fiber curves off to the right and the compression deformation increases more rapidly, the neutral axis lowers slowly and the steel deformation line continues straight. The final failure, due to crushing of the concrete at the upper surface, occurs before the deformation of the steel has reached that of the yield point of the steel and the final load is the maximum load. Fig. 171, shows the deformation in this case.

These experiments should not be considered to indicate that the steel should be stressed below point A, but rather that the tensile resistance of the concrete should not be included when figuring the resisting moment ot the beam.

As regards the allowable working stress on the steel reinforcement in the light of the experiments of Profs. Hatt, Talbot and Turneaure, it appears that on the average the point A corresponds to a stress in the steel of from 3,000 to 5,000 lbs. per sq. in. and that the first visible crack will not appear until a stress has been reached of from 12,000 lbs. in Prof. Turneaure's, 23,300 lbs. in Prof. Talbot's and 27,000 lbs. in Prof. Hatt's tests. It is therefore not necessary to confine the stress in the steel under the worst possible condition to less than 5,000 lbs. per sq. in. and the usual practice of allowing working stresses as high as 16,000 lbs. per sq. in. will not, under normal conditions, give dangerous cracks in the bottom of the beam.

Methods of Failure of Beams Under Tests.—The discussion of various methods of failure of beams under test by Prof. A. N. Talbot, in Bulletin No. 4 of the University of Illinois (April 15, 1906), sets forth so clearly the results of study of carefully con-

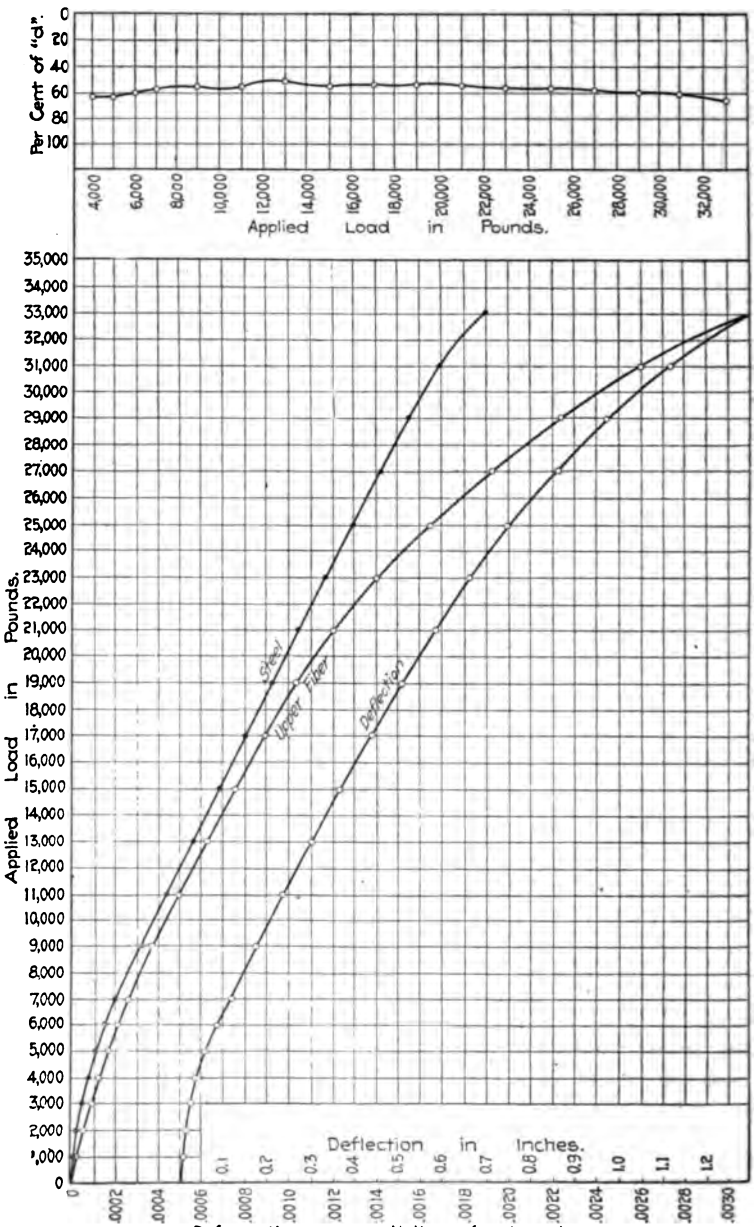


Fig. 171.—Diagram Showing Change in Neutral Axis and Deformation of Test Beam.

ducted beam tests that it seems desirable to insert a portion of this discussion in this place. The author is indebted to Prof. Talbot for permission to use this material.

Prof. Talbot states that in general a reinforced concrete beam may fail by one or more of the following methods: "1. Tension of steel; 2. Compression of concrete. 3. Shearing of concrete; 4. Bond or slip of bars; 5. Diagonal tension of concrete; 6. Miscellaneous methods, like the splitting of bars away from the concrete, the effect of the bearings, etc. What one of these methods of failure will govern the strength of a beam is dependent upon percentage of reinforcement, kind of steel, quality of concrete, relation of depth of beam to length of span, disposition of reinforcement, and other conditions.

"The stress which reaches the limit of the resisting property of the material is the one which will control the strength of the beam. It is not likely that two or more of these stresses will reach their point of failure at the same time. It is not even generally feasible so to proportion a beam that its strength shall be the same in tension, compression, bond and diagonal tension. For other reasons the amount of reinforcement or depth of beam may be made the same in spans of different length, or carrying different loads, and such a variation will change the relative value of tension compression, bond, etc. While it may be well to calculate the various stresses, in many cases the relative dimensions and amount of reinforcement are such that the method of failure may be told without much calculation.

"**Primary and Ultimate Failure.**—In judging the results of tests a distinction must be made between primary failure and ultimate failure. Some change or failure may take place in the beam during the test which will greatly modify the conditions, and we may not properly judge of the conditions existing at this time by what happens later. This early or critical failure may be named the primary failure, and its cause should be called the cause of failure of the beam. Thus slipping of the bars may come after diagonal failure has occurred. It is not always possible to know positively the cause of failure, but generally a careful study of the test will give a trustworthy conclusion.

"**Failure by Tension in Steel.**—Beams having shallow depth as compared with their length and having a moderate amount of reinforcement may, when tested with the usual way of loading, be expected not to fail before the steel has been stretched to its yield point, and the maximum load carried will generally be but little higher than that carried when the yield point is reached. Fig. 172 illustrates the typical form of failure by tension in steel. It should be noted that the tension cracks shown in the figure will appear considerably before the steel reaches its yield

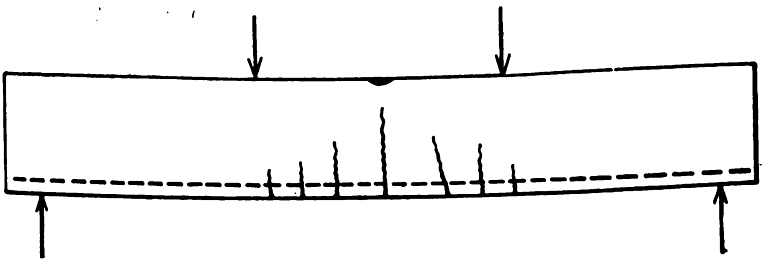


Fig. 172.—Beam Failure by Tension in Steel.

point. With other forms of failure these cracks may appear, but they do not grow to the extent they do in tension failures.

"**Failure by Compression of Concrete.**—Beams having a large amount of reinforcement may fail by the crushing of the concrete at the top of the beam before the steel has been stressed to its elastic limit. As has been stated, the amount of reinforcement

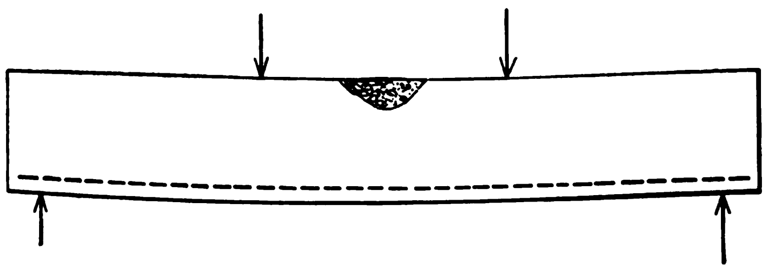


Fig. 173.—Beam Failure by Compression in Concrete.

necessary to develop the full compressive strength of the concrete depends upon the quality of the concrete and the elastic limit of the steel, and will vary from 1 to 1½ per cent. Figure 173 illustrates this form of failure. If stress-deformation diagrams are made, the line showing the shortenings of the upper fibre of the concrete will curve away rapidly from the usual straight-line position, but the steel deformation line will not be modified materially until near the line of failure. This condition of the stress deformation curves is the best evidence that the crushing strength of the concrete has been reached without developing the strength of the steel at the yield point.

"**Bond or Resistance to Slipping of Reinforcing Bars.**—In order to have beam action there must be a proper web connection between the tension and the compression portion of the beam. Where there is no metallic web reinforcement the concrete of the beam acts as this web. Of course, the amount of stress in the reinforcing bars and also in the compression area of the concrete varies at different cross sections along the length of the beam. The increment between consecutive sections of increase in the tensile stresses of the reinforcing bars is transferred to or connected with the increments of the compression stresses of the concrete by means of this web. In transmitting the increment of tension from the reinforcing rods to the surrounding concrete there is developed a tendency of the rods to slip in the concrete, and the amount of resistance to slip thus developed is called bond, and will be measured in terms of the area of surface in contact with the concrete. It will be seen that the total bond developed on the surface of the bars in one inch of length is equal to the total change in total tensile stress in the bar for the same inch of length. Bond may be compared to the action of the rivets joining flange to web in a riveted steel plate girder, except that in the reinforced concrete beam the contact is continuous.

"For horizontal reinforcement the formula for bond may be derived as follows: For any vertical section of the beam the equation $M = A_s f_s d'$ gives the resisting moment. (M = moment, A_s = area of steel, f_s = stress in steel, d' = distance from center of steel to center of pressure in concrete.) Differentiating this equation,

$$\frac{dM}{dx} = A_s \frac{df_s}{dx} d'$$

By the principles of mechanics of beams

$$\frac{dM}{dx} = V,$$

where V is the total vertical shear at the given section (reaction at support minus load between the support and the section considered). Substituting and transposing,

$$\frac{A_s\, d\, f_s}{d\,x} = \frac{V}{d'}.$$

Now the derivative $\dfrac{A_s\, d\, f_s}{d\,x}$

expresses the rate of change of the total tensile stress in the reinforcing bars at the section under consideration; it is given in terms of a unit of length of beam (lb. per inch of length), and measures what is transmitted to the concrete by bond. Using m as the number of bars, o as the efficient circumference or periphery of one bar, the total surface of bar for one inch of length of beam is mo, and the bond stress developed is mou, where u represents the bond developed per unit of area of surface of bar. Equating this to the value of the derivative given in the above equation, and solving

$$u = \frac{V}{mo\, d'}.$$

This equation is not applicable in just this form when the bars are bent up or inclined from the horizontal, since in this case d′ is a variable, and this fact will modify the differentiation.

"**Failure of Bond Between Steel and Concrete.**—Failure by the breaking of the bond between the steel and concrete is unusual for a beam having the proportions of ordinary test beams. The calculated bond stress developed in the beams tested at University of Illinois in 1905 ranges from 70 to 193 lbs. per sq. in., and the bond tests on plain mild steel rods give values from 200 to 500 lbs. per sq. in., and on some forms of deformed bars from 300 to 1,000 lbs. per sq. in. Size of beams tested was 8 ins. wide, 11 ins. deep and 13 ft. long, with test spans of 12 ft. The center of steel reinforcement was 10 ins. below top surface. The concrete used was a 1:3:6 mixture. It is true that conditions under which the bond tests are made differ from those in the beam, and also that bond stresses may not be distributed in the beam exactly as assumed, and considerable allowance should be made for these. Besides, the effect of time and of repetition of stress upon bond resistance is not known. For bars bent up out of the horizontal a much higher stress is brought into action near the end of the bar than with the bars laid horizontally throughout the length of the beam. The value of the bond resistance will depend upon the smoothness of the surface of the bar, the uniformity of its diameter, the adhesive strength of the concrete, and the shrinkage grip developed in setting. In most of the failures reported to be caused by slipping of the bars, it seems certain that this slipping occurred subsequent to diagonal tension failures or other changes which were the primary causes of failure. For mild steel reinforcement placed horizontally in beams of ordinary dimensions, the diagonal tensile strength of the beam will be a much weaker element than the bond stress between steel and concrete.

"Failure of the bond between the reinforcing rods and the concrete is difficult to detect. The fact that a rod has been found after failure of the beam to have slipped is not evidence that slipping occurred before failure began and hence was the primary cause of failure. In some instances reported as failure by slipping, the slipping evidently occurred as a consequence of the new conditions brought into play by whatever was the primary cause of failure, and slipping may not be considered the primary failure.

"A number of beams reinforced with tool steel rods having a smooth, almost polished surface were tested. All these beams failed by slipping of the bars. Their appearance after failure is shown in Fig. 174.

TABLE LXII.

VALUES OF VERTICAL SHEARING STRESS AND BOND DEVELOPED IN BEAMS REINFORCED WITH TOOL STEEL.

Beam No.	Vertical Shearing Stress lb. per sq. in. $v = \frac{V}{bd'}$	Bond lb. per sq. in. of surface of bar $u = \frac{V}{mod'}$	Remarks.
49	95	161	1.10% reinforcement.
53	72	123	1.10% reinforcement.
57	66	112	1.10% reinforcement.
60	107	181	1.10% reinforcement.
62	73	124	1.10% reinforcement.
61	73	124	1.10% reinforcement.
51	101	114	1.66% reinforcement.
52	126	143	1.66% reinforcement.
55	107	120	1.66% reinforcement.
Av.	91	133	
61	61	69	1.66% reinforcement.
56	...	...	1.10% reinforcement.

"Table LXII. gives the bond developed in lbs. per sq. in. at the time of failure, as calculated by the equation

$$u = \frac{V}{mo\ d'},$$

and also the vertical shearing stress developed with the same load. The weight of the beam and loading apparatus is included in these calculations. Beam No. 49 failed suddenly. The failure

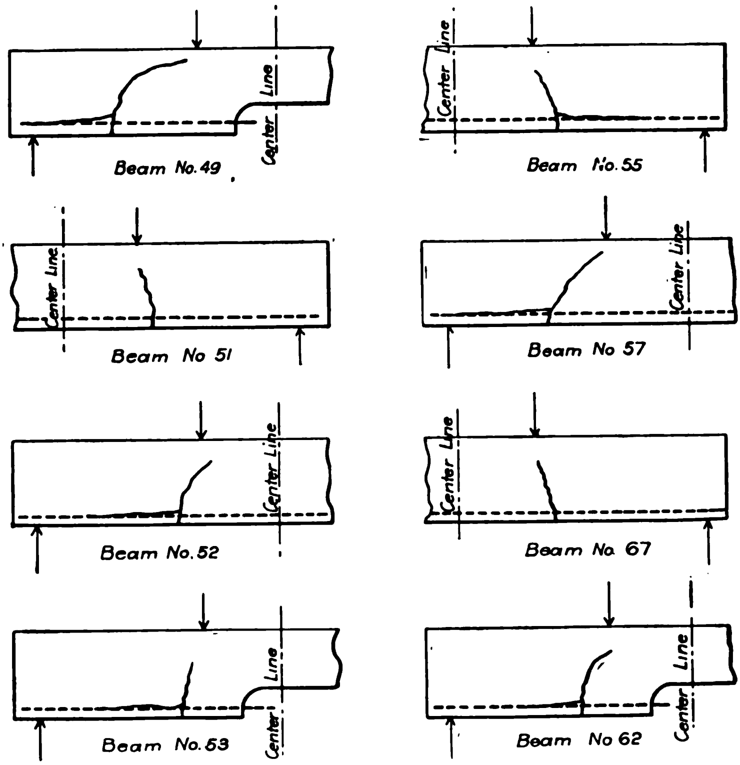


Fig. 174.—Failures of Beams Reinforced with Tool Steel.

shows a nearly vertical crack with a horizontal crack extending along the plane of the reinforcement toward the support. It seems likely that slipping occurred from the end of the rods to the vertical crack, and also that the horizontal crack developed at the time of slipping and in connection with the vertical tension coming on the rod. The bond stress developed, 161 lbs. per sq. in. of surface of bar, is the largest except one developed in this series. The vertical crack was closer to the support than was the case with the other beams.

"Eight beams may be described as slipping and failing gradually. At a load of 75 per cent. to 95 per cent. of the maximum, a crack, vertical or nearly vertical in position, appeared between the load point and the support and not very far from the former, and gradually increased in height until the maximum load was reached. The load then fell off, and this crack grew until suddenly failure occurred at a load from 1,000 to 4,000 lbs. less than the maximum. In beam No. 52 the critical crack appeared at 13,000 lbs., 87 per cent. of the maximum load. The direction and position of the critical crack are indications that slipping of the rods was the primary cause of failure. The cracks, as shown in Fig. 174, are as they appeared near the time of final failure. At first appearance only the vertical portion showed. It seems likely that this slipping occurred from the crack to a point under the load, there being no shear and hence no bond stress on the portion of the beam between the two loads at the third points of the beam. The calculated bond stress at maximum loads for these beams ranged from 114 to 143 lbs. per sq. in. Bond tests with this tool steel, the rods being embedded 6 ins. in the concrete, gave values of 153, 147, 154 and 141 lbs. per sq. in. of surface, averaging 149 lbs. per sq. in. The critical crack in these eight beams first appeared when the bond stress developed ranged from 90 to 125 lbs. per sq. in. The position of the critical crack and the manner of failure of this group of beams are materially different from the conditions accompanying diagonal tension failures. It must not be overlooked, however, that the presence of this initial crack does weaken the resistance of the beam to diagonal tension, and thus increases the web stresses above the crack and also the vertical tension transmitted from the rod just beyond the crack, which together cause the final failure to be of the form shown.

"There are two types of bond failures: (1) Slip from the direction of the middle of the span, with a slowly developing crack slightly inclined from the vertical, which extends upward as the load is increased to the maximum load, growing still more as the test is continued at a dropping load, and finally breaking by splitting below and cracking diagonally above. (2) Slip from the end of the beam and a sudden failure at maximum load by the formation of a crack slightly inclined from the vertical and near to the support, together with accompanying splitting and diagonal cracking at the top of the beam. The characteristic of the first is slow failure along a crack which is nearly vertical, and which gradually grows with increasing load, and of the latter a sudden failure through a crack in a nearly vertical position not visible until time of failure is reached. It is likely that both are variations of a single form of failure, the former appearing when the vertical tensile strength of the concrete is exceeded. In failure by diagonal tension, the cracks formed are inclined more from the vertical than are these cracks. In none of the tests made with mild steel bars placed horizontally was there any evidence of slip of bar, although in one beam a bond stress of 193 lbs. per sq. in. was developed.

"**Vertical and Horizontal Shearing Stresses.**—It is shown in the mechanics of beams that there exists throughout a beam vertical and horizontal shearing stresses which vary in intensity, and that at any point in a beam the vertical shearing unit-stress is equal to the horizontal shearing unit-stress there developed. As noted under bond, the total tension in the reinforcing bars varies along the length of the beam, as does also the total compressive stress. The horizontal shearing stress may be considered to transmit the increments or increase of the total tensile stresses in the reinforcing bars (which is transmitted to the surrounding concrete by the bond stresses) to the corresponding increments of compression in the compression area of the concrete, the concrete thus forming the stiffening web of the beam. The amount of this horizontal tensile stress so transmitted from the reinforcing bars per unit of length of beam is by the equation

$$mou = \frac{V}{bd'}.$$

"Consider this distributed over a horizontal section just above the plane of the bars for a unit length of beam, and call the horizontal unit stress v. The shearing resistance per unit of length of beam thus developed is then b v, and equating this to m o u,

$$v = \frac{V}{bd'}.$$

"This equation gives the horizontal shearing unit-stress, and therefore also the vertical shearing unit-stress, at a point just above the level of the reinforcing bars. As no tension is here considered as acting in the concrete, there will be no change in the intensity of the horizontal and vertical shearing stresses between this line and the neutral axis. For the part of the beam where tensile stresses extend well down to the reinforcement some modification of this treatment may be made. Above the neutral axis the intensity of the shearing stresses will decrease by the law of change of horizontal shearing stresses for homogeneous rectangular beams modified to suit the parabolic stress deformation relation. The distribution of the intensity of the

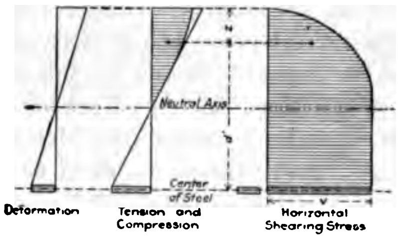


Fig. 175.—Distribution of Horizontal and Vertical Shear.

horizontal shearing stress over a vertical section is represented in Fig. 175.

"As d′ generally will not vary far from .85d, the shearing stress by equation

$$v = \frac{V}{bd'}$$

will be, say, 18 per cent. more than if considered to be uniformly distributed over a vertical section extending down to the center of the reinforcing rods. Even if tension is considered to exist in the concrete for a short distance below the neutral axis, the shearing stress will not be greatly modified thereby. If the bars are inclined or bent up from the horizontal, the above equation must be changed to allow for a variable d′.

The horizontal and vertical shearing unit-stresses obtained by the use of the above equation $v = \dfrac{V}{d'}$

are low, the highest value developed for the beams tested by Prof. Talbot being 151 lbs. per sq. in. Even if we consider a point in a beam at which the concrete is carrying stress in tension up to its ultimate strength, the value of the diagonal shearing will scarcely reach twice the vertical shearing stress. The shearing strength of concrete is much higher than this, probably from 50 to 75 per cent. of the compression strength. As a rule, reinforced concrete beams do not fail by shear. What has been called shearing failures are really diagonal tension failures.

"**Diagonal Tension in the Concrete.**—In the flexure of a beam stresses are set up in the web which are sometimes called web stresses and sometimes secondary stresses. Besides the horizontal and vertical shearing stresses already discussed, tensile or compressive and shearing stresses exist in every diagonal direction. In determining the bending moment only the horizontal components of these are taken. When there is no metallic web reinforcement all the diagonal stresses are taken by the concrete. By the analysis of combined shear and tension the value of the maximum diagonal tensile unit stress (see Merriman's Mechanics of Materials, p. 265, 1905 edition) is found to be

$$t = \tfrac{1}{2} s + \sqrt{\tfrac{1}{4} s^2 + v^2},$$

"When t is the diagonal tensile unit-stress, s is the horizontal tensile unit-stress existing in the concrete, and v is the horizontal or vertical shearing unit-stress. The direction of this maximum diagonal tension makes an angle with the horizontal equal to one-half the angle whose cotangent is $\tfrac{1}{2}\dfrac{s}{v}$.

"If there is no tension in the concrete, this reduces to $t = v$, and the maximum diagonal tension makes an angle of 45° with the horizontal, and is equal in intensity to the vertical shearing stress.

"When the diagonal tensile stresses developed become as great as the tensile strength of the concrete, the beam will fail by diagonal tension, provided there is no metallic web reinforcement. Fig. 176 gives the typical form which this failure takes. As the value of the maximum diagonal tensile stress developed in a beam is by equation

$$t = \tfrac{1}{2} s + \sqrt{\tfrac{1}{4} s^2 + v^2},$$

dependent upon the horizontal tensile stress developed at the same point, it is difficult to compute its actual amount. The best method seems to be to compute the horizontal and vertical shearing unit-stress, and make all comparisons on the basis of this value. Beams which fail by diagonal tension and which are without metallic web reinforcement give a value of 100 to 150 lbs. per sq. in. for the vertical shearing unit-stress when calculated by the equation

$$v = \frac{V}{bd'}$$

(and lower values for poorer concretes), the limit depending upon the strength of the concrete. When these values are combined in the equation

$$t = \tfrac{1}{2} s + \sqrt{\tfrac{1}{4} s^2 + v^2},$$

with the probable horizontal tensile stress developed in the concrete below the neutral axis, the resulting diagonal tensile

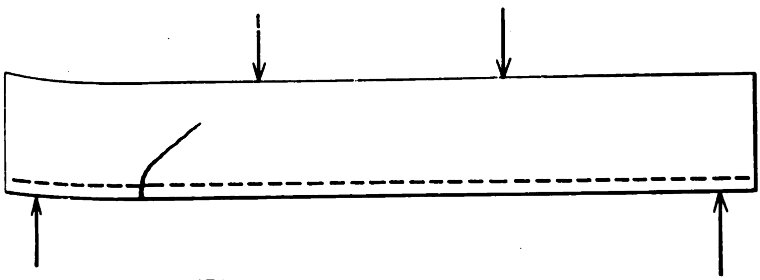


Fig. 176.—Beam Failure by Diagonal Tension.

stress is evidently the full tensile strength of the concrete. Diagonal tension failures are frequently characterized by sudden breaks, without warning, as is the case in the failure of plain concrete beams. A variation from this gives a slower failure, part of the shear being carried through the reinforcing bars and the ultimate failure involving the splitting and stripping of the bars from the beam above.

It is evident, since the vertical or external shear is independent of the resisting moment, that the relation between the depth and length of a beam will determine whether the beam will fail by diagonal tension or by tension of steel or compression of concrete. In relatively short and deep beams the diagonal tensile strength will fix the strength of the beam, while in long, shallow beams this element may be disregarded.

Since the diagonal tension may be resolved into horizontal and vertical or other components, the concrete may be relieved of a part of the diagonal tensile stress by one or both of two means: (1) By bending the reinforcing rods or strips sheared from them into a diagonal position, and (2) by making use of stirrups to take the vertical component of the diagonal tension." (The various methods of providing for these shearing stresses are discussed in another chapter).

"**Failure by Splitting of Bars Away from Upper Portion of Beam.**—Failures sometimes occur, either after a diagonal crack has appeared or at the same time that such a crack is observed, in which the reinforcing bars and the concrete below the level of the bars are split away from the remainder of the beam, the crack running horizontal for some distance. This stripping is caused by vertical tension in the concrete transmitted to it by the stiffness of the reinforcing bars after the concrete fails to carry its assignment of diagonal tension. In Fig. 177 consider that

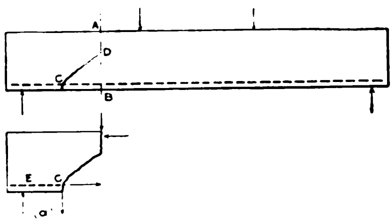


Fig. 177.—Failures When Bars Split Away from Upper Portion of Beam.

a diagonal crack, C D, has been formed. Take a vertical section through A D. On account of the diagonal crack normal beam action does not exist, and part of the vertical shear from the main portion of the beam is transmitted by the projecting portion of the beam D C B acting as a cantilever, and the flexural stiffness of the bars to the point C, and there applied to the left portion of the beam as a downward force. Figure 177 (a) shows the part at the left of A D acting as a free body. The part of the vertical shear applied at C tends to split the bars from the beam, starting at C and running toward E. This action is resisted by the tensile strength of the concrete in a vertical direction, and when this is exceeded the bars will split from the concrete above. This may happen without any horizontal movement or slip of the bars. Splitting of bars from the beam presupposes a failure in diagonal tension, for as long as true beam action exists vertical tension is not developed. After the diagonal crack is formed this part of the beam takes on the nature of a truss. This form of failure is then a secondary failure, though under some conditions the load carried before splitting occurs may be considerably more than that at which the diagonal crack appeared. This explanation shows why the concrete at the bottom of the bars continues to adhere to the bars. There is no evidence of shearing failure in these cases."

Attention should also be called to the danger from spacing bars too closely or with not sufficient concrete below the bars.

Tests were made to determine the effect of various kinds of loadings, repeated application of load, the effect of rest after release of load and effect of retention of load. Details of these tests can not be given in this place, but the conclusions drawn by Prof. Talbot are as follows:

"Center loading may be expected to give results which are higher than those found by the ordinary beam formula. Moments of resistance derived from results of center loading tests may not properly be used as a basis of calculation for other forms of loading. The results with loading at the one-third points compare favorably with multiple-point loading, and are comparable with uniform and other distributed loading.

"Repeated applications of a load which sets up high compressive stresses in the concrete give increasing deformations. The deflections after ten to fifteen applications were found to be 12% to 30% in excess of the deflection at the first application.

"Beams which were loaded to give a stress of 15,000 lbs. per sq. in. in the steel and 800 lbs. per sq. in. in the concrete, or more, failed to return to their original position upon the removal of the load, the amount of the retained deflection being 20% to 35% of the deflection. No appreciable recovery of the set was apparent after periods of 15 to 40 hours.

"Beams loaded so as to develop stresses of 18,000 to 32,000 lbs. per sq. in. in the steel and compressive stresses of 800 to 1,400 lbs. per sq. in. in the concrete gave little perceptible change in appearance or growth of cracks after the load had been retained 20 to 38 hours, and upon the application of greater loads the load-deformation curves and deflection curves rose upward and took the general shape for such curves for progressively applied loads. During the retention of load, the deflection increased 12%

to 35%, the principal cause of this increase evidently being the increased compression of the concrete."

Position of Neutral Axis.—Observations were taken to determine the position of the neutral axis throughout the different stages of flexure. During the first stage it remained below the middle of the beam, during the second stage the axis usually rises and then remains in one position during the third stage until the maximum load is nearly reached. Beyond the maximum load for a normal amount of reinforcement the axis rises somewhat higher during the fourth stage, but with the excess of reinforcement it lowers during the rapid deformation of the upper fibre.

During the third stage the position of the neutral axis did not differ materially for different forms of reinforcement. In general the neutral axis was found to be apparently lower than is given in several theories. Prof. Talbot states that the following formula locates fairly accurately the position of the neutral axis for

$$\frac{E_s}{E_c} = 10.$$

$$x = 0.23 + 0.16\ p,$$

x being the proportional depth of the neutral surface, and p the percentage of steel or the ratio of area of steel to the area of concrete, the depth from the top of the beam to the center of the steel being used in both cases.

Conservation of Plane Sections.—To determine if the usual hypothesis that plane sections before bending remain plane sections after bending held true, Prof. Talbot made extensometer measurements on two similar beams, 13½ ins. deep, with the reinforcement 12 ins. below the top face. On one of the beams the contact points were 11 ins. and 8½ ins. apart vertically, the upper point of the extensometer in each case being 1¼ ins. below the top of the beams. In the second beam the points were 11 and 6 ins. apart vertically, with the upper extensometer point as before. On each beam the readings of the extensometers were taken simultaneously as the loadings of the beams progressed. From these observations two sets of values of the elongation of the steel and compression of the concrete were determined and also the resulting positions of the neutral surface. Prof. Talbot states that the results agree closely, perhaps as closely as the variations in the transmission of the interior deformations, to the contact point, could be expected to agree.

Distribution of Stresses in a Beam.—There is considerable difference of opinion in regard to the elastic behavior of the concrete in a beam subject to flexure. As has been explained, the coefficient of elasticity, that is, the ratio of the stress to the deformation, is not a constant quantity, but decreases as the load brought upon the concrete increases. The ordinary theory of flexure is based upon Hooke's law, which is that the ratio between the stress and the strain or deformation for a given material is constant within its elastic limit. It has, however, been found that this law holds true for concrete only when the stresses are quite low.

The variations in the resistance of the successive fibres of the concrete above and below the neutral axis depend upon the conservation of plane sections. As has been stated Profs. Talbot

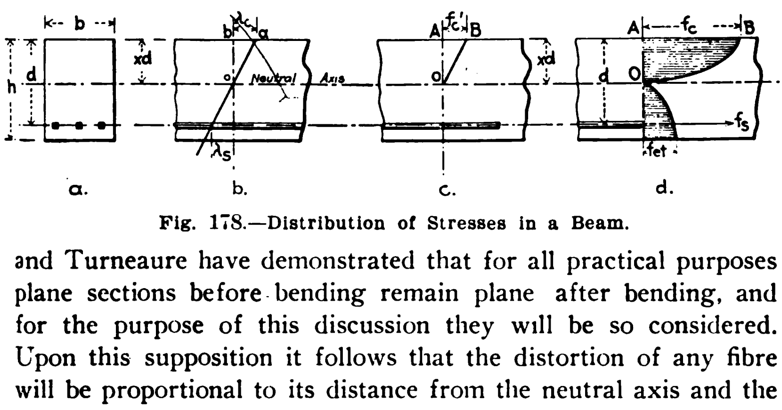


Fig. 178.—Distribution of Stresses in a Beam.

and Turneaure have demonstrated that for all practical purposes plane sections before bending remain plane after bending, and for the purpose of this discussion they will be so considered. Upon this supposition it follows that the distortion of any fibre will be proportional to its distance from the neutral axis and the law of variation of compression stress will be represented by some curve which closely approximates a parabola with its vertex at the fibre of greatest compression. It is upon the form of this curve that authorities do not agree. When its exact form is known the laws governing the relation between the elastic deformation, the stress and the modulus of elasticity, will become known.

Let Fig. 178(a) represent the cross section of a beam under flexure. Then Fig. 178(b) will represent the elastic deformations following the theory of conservation of sectional planes. Fig. 178(c) will show the conditions when the stresses are so small that the modulus of elasticity of the concrete may be considered as constant.

In this stress diagram the triangle A O B represents the total compressive stress on the concrete for width of beam b, and as f_c' represents the maximum intensity of the stress, the total stress acting will be represented by

$$\frac{f_c' x d\, b}{2}$$

and, as its center of gravity is $\frac{2}{3} x\, d$ above the neutral axis, its moment about this axis is

$$\frac{f_c' x d\, b}{2} \times \frac{2}{3} x d; \text{ or, } M = \frac{f_c' b\, x^2 d^2}{3}.$$

The conditions, which are considered to govern the action of the compressive stresses when the modulus of elasticity is assumed to be constant throughout the whole range of stress, are indentical with those given above. Such a distribution of stress is usually termed the rectilinear relation between stress and strain.

Stress Under a Varying Modulus of Elasticity.—It is almost universally admitted that the coefficient of elasticity varies with the loading. This is undoubtedly true, as it has been demonstrated by a large number of tests. In spite of this the author has seen stated by a recent writer that, as a result of recent experiments made by him, the modulus seems to be constant. A study of his tests brings to light the fact that his experiments were made on a carefully balanced, extremely dense concrete and only represent the values of the modulus under exceptional conditions. Under normal conditions concrete will be far from as well proportioned or dense, and the curve of compression will be more truly represented by a parabola or some other curve.

In Fig. 178(d) the extreme fibre is supposed to be subjected to a stress f_c, the fibres nearer the neutral axis, have a smaller stress per square inch, and the modulus of elasticity for each smaller stress is greater than of that for the fibre next above it and nearer the top of the beam, but as has been stated, in order that a section, which is plane before bending, shall remain plane after bending, the strain must be proportional to the distance from the neutral axis. It follows that the stresses on the inner fibres do not decrease according to a rectilinear law as represented by the triangle of stresses, but are greater than are indicated by the ordinates of a triangle. If the curve O B is assumed to be a parabola, the total stress on the concrete above the neutral axis will be represented by the area within the parabola. This is $2/3\ f_c\, xd$, and the total compression on the section of the beam, whose width is b, will be $2/3\ f_c\, xd\, b$. The center of gravity of the parabola is $5/8$ xd above the neutral axis, and the moment of the total compressive stress on the beam will be $2/3\ f_c\, xd\, b \times 5/8\ xd$, or $M = {}^5/_{12}\ f_c\, b\, x^2 d^2$.

M. Considère considers that this parabolic distribution of stress represents closely the action of the internal stresses in the cross-section of a beam under flexure. The studies of Profs. Hatt, Howe, Talbot, Turneaure and others upon compression tests of 12-in. cubes made at Watertown Arsenal confirm M. Considère's assumption, showing that the curve of compressive stresses closely approximates a parabola with its origin at the extreme outer fibre of the concrete, and its axis perpendicular to the direction of the applied forces.

Capt. John S. Sewell has made a special study of the Watertown Arsenal tests, plotting the curves of a large number of tests, and states in a paper to the International Engineering Congress at St. Louis, in 1904, that while curves of individual tests were often quite irregular in form, by grouping and combining curves according to the strengths of the concretes, they became more regular, and that for the leaner mixtures they became quite regular. He suggests that the great variation in the specimens of rich concrete is probably due to shrinkage strains in setting. Details of Capt. Sewell's studies can not be given here, but the results are as follows: The curve of stress was found to lie between the straight line of the triangle representing the rectilinear distribution of stress and the curve representing the parabolic distribution. He found that the areas included between the axis of coordinates and curves plotted from actual tests were about 20 per cent. in excess of the area representing the triangular distribution, while a parabolic representation is $33^1/_3$ per cent. in excess of the triangular areas, and the height of its center of gravity is between the heights $1/2$ xd and $2/3$ xd of the triangular and parabolic areas respectively. He, therefore, assumes its area to be $5/8\ f_c\, xd$, and the height of its center of gravity to be 3/5 xd. The compressive stress upon the cross-section of the beam will, therefore, be $\frac{5}{8}$ f_c xd b, and its moment about the neutral axis $\frac{3}{8}$ $f_c\, b\, x^2 d^2$. This assumption probably gives a value of the stress nearer its true value than either of the other two methods outlined above, and would appear to give conservative results when used in the design of beams.

Elongation or Stretch of Concrete in a Reinforced Beam.—In general the concrete in a reinforced beam stretches similarly to the concrete in a plain beam. In the latter case, however, the beam ruptures when the limit of stretch, sometimes called the elastic limit, is reached. Rupture does not occur, however, in a reinforced beam when the limit of the tensile strength of the concrete in the lower part of the beam is reached. In fact the stress often passes considerably beyond the rupture point of a plain concrete beam before visible cracks appear. This is probably due to the action of the reinforcement in distributing the stress over the entire length of the piece, while in non-reinforced beams the stretch is localized and rupture takes place.

Considère advanced the theory that the concrete in tension in a reinforced concrete beam can be strained far beyond its elastic limit, or far beyond the breaking point of non-reinforced concrete. He bases this theory on experiments made by him, in which he found that reinforced beams took a deflection under transverse loading far beyond the point at which a beam not reinforced would break without any apparent injury to the concrete on the tensile side of the beam. The usual amount of stretch of elongation, which plain concrete will undergo before rupture, is about 0.0001 part of its length. Considère was not able to discover cracks or evidences of failure in some of his specimens when the stretch was 0.001 part of their length, and is said to have cut sections of concrete out of the beam below the reinforcement after having subjected it to such a stretch, and upon testing them in tension, found them to have the same tensile strength possessed by plain concrete specimens which had not been subjected to stretching. While recognizing the value of tests made by this eminent authority, the acceptance of the above theory without evidence from similar tests made by other experimenters would seem unadvisable. Such a theory would indicate that the combination of the two materials possesses physical properties different from those shown by concrete, when considered by itself. It is improbable that such is the case. In fact, Considère's theory is not borne out by later experiments conducted by Profs. Talbot, Turneaure and others.

Prof. Turneaure made tests upon reinforced beams which had been kept under water until they were tested. By placing the beams in water, as soon as they were hard enough to do so, prevented the formation of hair cracks, due to contraction in setting. The beams, while wet, were tested with the tensile side up by applying the load from below. This enabled more careful observations to be taken. It was found by repeated tests that when the flexure was such that the unit elongation of the concrete was between 0.0001 and 0.0002 narrow bands of moisture, perhaps ⅛-in. wide or water marks, appeared on the surface at some places on the tensile side, the moisture apparently coming through cracks. That they were actual cracks at these water marks was proved by sawing out a strip of concrete containing such a mark. In all cases the strips fell apart at the water mark. Strips cut between water marks on the contrary were uninjured.

As the flexure was increased the marks developed into visible dark hair-like cracks, and at a unit elongation at or above 0.00035 were plainly visible to the naked eye. We may infer from these tests that M. Considère did not happen to include a crack in the test pieces cut from the tension side of his beam. While it is possible his beam did not crack, Prof. Turneaure's tests show that cracks at times do occur, and it is probable that they always occur when the stress in the tension side of the beam exceeds the tensile strength of the concrete. Prof. Talbot showed by his tests that the concrete and metal act together as a homogeneous material until a unit fibre elongation of from 0,0001 to 0.000133 is reached, when the concrete seems to lose its tensile strength and its load is thrown upon the steel. The steel then begins to elongate more rapidly and fine hair cracks begin to appear when the tensile stress in the steel becomes about 15,000 lbs. per sq. in.

In plain concrete no water marks or cracks were observed before rupture. A comparison of the observed and calculated elongations of the reinforced concrete with those for plain concrete at rupture, shows that the initial cracking in the former occurs at an elongation practically the same as in the latter. Prof. Turneaure writes as follows in regards to this phenomenon:* "The significance of these minute cracks is an open question. It has been supposed that concrete reinforced by steel will elongate about 10 times as much before rupture as plain concrete. These experiments show very clearly that rupture begins at an elongation about the same in both cases. In the plain concrete total failure ensues at once; in the reinforced concrete, rupture occurs gradually, and many small cracks may develop, so that the total elongation at final rupture will be greater than in plain concrete. In other words, the steel develops the full extensibility of a non-homogeneous material, that otherwise would have an extension corresponding to the weakest section.

"The presence of these cracks, of course, seriously affects the tensile strength of the concrete, and as they appear at an elongation corresponding to a stress in the steel of 5,000 lbs. per sq. in. or less, it would seem that no allowance should be made for the tensile resistance of the concrete. Furthermore, if such cracks are present the calculation of the tensile resistance of reinforced concrete by the method used by Considère leads to no useful result. In his tests Considère determines the stress in the steel from measurements of its elongation and then assumes the concrete to carry the remainder. Assuming the value of E to be uninfluenced by the concrete, this would be correct, so long as the stress in the steel and in the concrete is uniform between points of measurement. As stated by Considère himself, such results are only *average* values. But concrete may be cracked entirely through and yet possess a very considerable *average* tensile strength over a length of several inches. Obviously in that case an average is of no value; the strength of the concrete is usually taken at zero."

In practical design the most important question which arises is how far a concrete may be cracked without exposing the steel to corrosive influences. In this respect it seems to the writer that the minute cracks, which appear in the early stages of the tests, can have very little influence. However, the entire question of the effect of the cracks and pores in the concrete on the corrosion of the steel needs careful investigation.

The question may well be asked if the presence of these minute

*Proceedings of Am. Soc. for Testing Materials, 1904.

cracks will prove dangerous to the steel. While reliable data on this subject are much needed to answer this important question, it is reasonable to assume that under ordinary conditions of loading the cracks are so small that no dangerous action will take place. Again, if the structure be occasionally strained up to or slightly above the assumed working stress of the steel, its elasticity is such that when this extreme loading is removed the cracks will close up again, expelling any moisture which may have collected.

The moisture and acid gases in the atmosphere are the active elements producing corrosion; that they will act in cracks so minute that they can not be detected by the unaided eye is improbable, especially when we consider that their action must take place in close proximity to an alkali like cement, which is present in the concrete, and in all probability covers the metal with a protecting film, even if the concrete be ruptured clear to the metal.

Care should be taken in the design of works to be subjected to extreme exposure in severe climates, to keep the unit stresses in the steel low, thereby avoiding entirely or reducing to a minimum possible cracking in the tension flange of the concrete.

If an approximate computation be desired for the elongation in the bottom or tensile side of a beam it may be obtained by the use of the formula

$$E = \frac{Pl}{A\lambda}; \text{ transposing we obtain } \lambda = \frac{Pl}{AE},$$

in which λ is the deformation required, l = the length of steel in tension, A = area of steel and P = stress in steel, which may be computed in the usual manner. E may be taken at 30,000,000. Inserting these numerical values in the formula and solving the required elongation λ will be obtained. Thus if $P = 30{,}000$ $l =$ 8 ft. 4 in. = 100 ins., $A = 2$ sq. ins., we have

$$\lambda = \frac{30{,}000 \times 100}{2 \times 30{,}000{,}000} = 0.05 \text{ inches.}$$

Tensile Resistance of Concrete in Reinforced Beams.—From the facts as above set forth in regard to the experiments of Profs. Talbot and Turneaure, we may safely conclude that the tensile strength of the concrete should be neglected in all computations for the strength of beams, with the exception of computations for the deflection of beams, when the stresses are quite low. This agrees very well with the practice of most of the leading engineers in this country as well as in Europe. The regulations of the Building Department of the Borough of Manhattan, New York City, the Prussian regulations for concrete buildings, issued by the Minister of Public Works of Prussia, and the regulations adopted by a commission of experts for the French Government to establish a building code for reinforced concrete structures, provide that the steel shall be considered as taking the entire tensile stress in beams. Hence, we conclude that all tensile stress in the concrete should be neglected and in the design of beams will assume that all tensile stress is carried by the steel.

CHAPTER XIX.

THEORY OF BEAMS.

Theory of Beams.—The theory of reinforced concrete has been the subject of much study by engineers and mathematicians for a number of years. Theoretical investigations in conjunction with practical tests have been made and much valuable data obtained, but, unfortunately, very little uniformity is shown in tests by different experimenters and discrepancies in results necessarily appear; nevertheless, much knowledge of the properties of this material of construction has been obtained. Within the past year or two a series of experiments has been undertaken by Profs. Talbot, Turneaure, Howe, Hatt and others, in which great care has been taken to secure uniformity of conditions. Some of the results of the tests, already available, have been given in the preceding pages and have done much to clear up doubtful points. The continuation of these experiments will undoubtedly do much to advance scientific knowledge on this subject.

As the main object of reinforced concrete is to secure a material which will withstand strains due to transverse loading, it is of prime importance to secure a theoretical formula or formulas for use in the design of the section of beams, girders and slabs at the point where the bending moment is a maximum. It is desirable, if possible, to secure a rational formula, but it is not absolutely essential to successful design that the formulas be rational, as empirical formulas, if properly applied, may, and do agree closely enough, for all practical purposes, with the results obtained from actual tests upon reinforced concrete pieces. Many such formulas are used in the design of reinforced concrete structures, yet, other things being equal, it is desirable to use the formula or formulas which embody most fully all conditions entering into the problem. The theory developed by Prof. Hatt, with certain modifications, seems to the writer the most rational of the numerous theories which have been advanced, and will be employed in this work for the design of beams, slabs and girders. The development of this theory is given in the succeeding pages. In another chapter a number of the theories most widely used in this country at the present time will be given, not because any one of them is more essential to the successful design of reinforced concrete than any other, but to show the best known present practice.

The present trend of thought on this subject seems to be to develop a rational formula and, after securing such a one, to determine as closely as possible the value of certain factors contained therein, inserting these values, neglecting other unimportant factors and then reducing and simplifying these rational formulas until they take the form of straight line formulas. Prof. Hatt's formulas, thus reduced, become very simple in application and are coming into great favor.

The theory of reinforced concrete pieces, strained in flexure, is usually based upon the following assumptions:

First. There is so perfect a union between the concrete and metal and the latter is so distributed that the two will act together as a practically homogeneous material.

Second. Sectional planes before bending remain plane surfaces after bending within the elastic limit of the steel.

Third. There are no initial stresses in either the concrete or the metal due to the shrinkage of the concrete in setting.

Fourth. The applied forces are parallel to each other and perpendicular to the neutral surface of the beam before bending.

Fifth. The values of the coefficients of elasticity obtained in direct tension and compression apply to the material under stress in beams.

Sixth. The entire tensile stress is carried by the steel.

First. As has been already stated, the utility and safety of reinforced concrete for structural purposes depends largely upon a close union between the two materials. When the adhesion is not great enough mechanical bonding should be used. In order that the stresses may be more easily transmitted from the concrete to the steel, it is desirable that a number of small bars uniformly spaced be used in preference to one or more large bars. This arrangement will cause the combination to approximate more nearly a homogeneous material.

Second. The hypothesis of conservation of plane sections is universally accepted, and, as has been explained, is probably approximately correct.

Third. As a matter of fact, there are always initial stresses due to expansion or contraction of the cement in setting, to change of temperature, amount of moisture present during setting, etc. M. Considère demonstrated, by means of tests on plain concrete, that when setting in water the specimens increased in length and when setting in air, contraction took place. The amount of change at the end of two or three years was probably from 0.0015 to 0.0020 of the length. The internal stresses are so uncertain and of so small importance that their effect is usually neglected in developing a formula for the strength of beams. Again an attempt to take them into account would introduce great complications without in the least diminishing the possible error.

Fourth. It is essential for the purpose of analysis that the applied forces shall be parallel to each other and perpendicular to the neutral axis; otherwise it will be necessary to resolve the forces parallel and perpendicular to the neutral axis. This will introduce another unknown quantity and greatly complicate the problem.

Fifth. It is assumed that the coefficient of elasticity of concrete is variable within the limits of stress and an assumption is always made as to the form of the stress-strain curve of concrete in compression. The form which the stress-strain diagram takes for a variable coefficient of elasticity has been explained in a preceding chapter. In this discussion we will first assume that the curve is a parabola and then give in a subsequent chapter the modifications necessary in the formulas when a rectilinear relation and the relation expressed by Capt. Sewell's curve are used.

Sixth. The results of all recent tests seem to indicate that the tensile strength of the concrete should be neglected, with the reservation that it shall be taken into consideration when calculating the flexure of a beam under moderate stresses. Again, as was explained in the preceding chapter, it seems advisable that the tension in the lower part of the beam shall not be so great that the elongation of the concrete shall exceed 0.001 of its length.

Let E_s = modulus of elasticity of steel.
Let E_c = modulus of elasticity of concrete in compression.
f_s = tensile stress in steel, lbs. per sq. in.
f_c = compressive stress in concrete, lbs. per sq. in.
A_s = area of steel in tension.
A = area of cross-section of concrete from the top face to center of reinforcement, $A = bd$.

λ_s = unit elongation of steel in tension.
λ_c = unit compression of extreme fibres of concrete in compression.
b = breadth of beam in inches.
d = depth of reinforcement below compression face of beam = effective depth of beam.
xd = distance of outside compression face of beam to neutral axis where d is depth of reinforcement.
$p = \dfrac{A_s}{A}$ = ratio of cross-section of steel in tension to cross-section of beam above center of gravity of steel.
$e = \dfrac{E_s}{E_c}$ = ratio of modulus of elasticity of steel to concrete.

We will assume a rectangular beam under flexure. Fig. 179

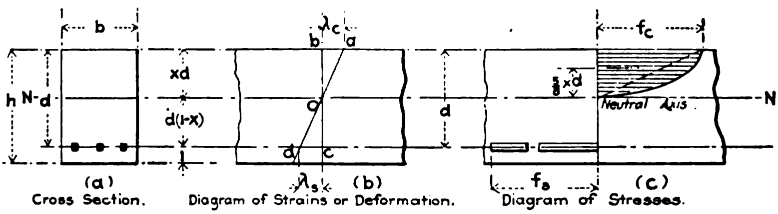


Fig. 179.—Beam Deformation and Stress Diagram.

a, b and c, shows a cross-section of the beam, and gives a graphical representation of the deformation and stresses. λ_c in Fig. 179(b) represents the deformation of the extreme fibre of the concrete in the compressive side of the beam, and λ_s the unit elongation of the steel. Since sectional planes before bending remain plane after bending.

$$\frac{\lambda_c}{\lambda_s} = \frac{xd}{(1-x)d} \qquad \text{(1).}$$

But

$$E_c = \frac{f_c}{\lambda_c} \text{ and } E_s = \frac{f_s}{\lambda_s};$$

or

$$\lambda_c = \frac{f_c}{E_c}, \text{ and } \lambda_s = \frac{f_s}{E_s}.$$

Therefore

$$\frac{\lambda_c}{\lambda_s} = \frac{xd}{(1-x)d} = \frac{f_c}{E_c} \times \frac{E_s}{f_s} \qquad \text{(2).}$$

Transposing and reducing

$$f_c = \frac{E_c x f_s}{E_s (1-x)} \qquad \text{(3).}$$

The total stress on the concrete above the neutral axis is represented by the area within the parabola, when a parabolic distribution of stress is assumed; this equals $2/3\ f_c\ xd$, for a unit width, and the total compression F_c on the section of a beam having a width b, is

$$F_c = \tfrac{2}{3} f_c xdb \quad \ldots\ldots (4).$$

The total tension F_s in the steel is

$$F_s = A_s f_s \quad \ldots\ldots (5).$$

For equilibrium these two forces, which act parallel to each other and in opposite directions, must be equal. They may be considered as the two forces of a couple and

$$A_s f_s = \tfrac{2}{3} f_c xdb \quad \ldots\ldots (6).$$

Dividing both sides of this equation by bd and remembering that $bd = A$, and $\dfrac{A_s}{A} = p$, we obtain

$$pf_s = \tfrac{2}{3} f_c x \quad \ldots\ldots (7).$$

Substituting for f_c its value from equation (3) we obtain

$$pf_s = \frac{2E_c f_s x^2}{3E_s(1 - x)},$$

reducing

$$\tfrac{2}{3} x^2 = \frac{E_s}{E_c}(1 - x)\,p \quad \ldots\ldots (8).$$

from which

$$x = -\tfrac{3}{4}\frac{E_s}{E_c}p \pm \sqrt{{}^3/_2 \frac{E_s}{E_c}p\left(1 + \tfrac{3}{8}\frac{E_s}{E_c}p\right)} \quad \ldots\ldots (9).$$

Or replacing $\dfrac{E_s}{E_c}$ by e.

$$x = -\tfrac{3}{4}ep \pm \sqrt{\frac{3}{2}ep\left(1 + \frac{3}{8}ep\right)} \quad \ldots\ldots (10).$$

When the values of e and p are assumed, the position of the neutral axis may be found by solving this quadratic equation. Finally, when the position of the neutral axis is known, the moment of resistance of the beam may be found. Taking the center of moments about the neutral axis the total resisting moment of the beam is equal to the sum of the moments of compression in the concrete and tension in the steel, and if M_r represents the resisting moment of the beam,

$$M_r = {}^5/_{12}\, f_c b x^2 d^2 + A_s f_s d\,(1 - x) \quad \ldots\ldots (11).$$

But in order that the beam shall not fail, the resisting moment must be equal to or greater than the bending moment. Then placing $M_r = M$, and reducing eq. 11 becomes

$$M = [{}^{5}/_{12}\, f_c x^2 + p f_s (1 - x)]\, bd^2 \quad \ldots\ldots\ldots\ldots\ldots (12).$$

The coefficient in this equation contains both f_c and f_s, and, therefore, the equation is not in a convenient form for use in the solution of beams, as it is usual, when a definite value of p has been determined for use under the given conditions, to assume a safe working value for either f_c or f_s, compute the section of the beam and then determine if the working value of f_s of f_c for this section, as the case may be, comes within safe limits. By substituting successively in eq. (12) the values of f_s and f_c from equation (7) and reducing equation (12) takes the form:

$$M = \tfrac{2}{3} f_c x\, (1 - \tfrac{3}{8} x)\, bd^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (13).$$

when the allowable stress of the concrete is assumed and

$$M = p f_s\, (1 - \tfrac{3}{8} x)\, bd^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (14).$$

when the allowable stress in the steel is assumed.

The coefficient $p\, f_s\, (1 - \tfrac{3}{8}x)$ will be found to be the determining factor when a low percentage of steel is used, or when a moderately low percentage of steel with a concrete of high strength is used.

The coefficient $2/3\, f_c\, x\, (1 - \tfrac{3}{8}x)$ will be the determining factor when a high percentage of steel is used, when a steel of high elastic limit is used or when a concrete of low crushing strength is used. These two coefficients become equal to constant quantities when definite values have been assumed for p, e, f_c and f_s. These coefficients may be represented by a constant K. Then K may have either of the values $K = 2/3\, f_c\, x\, (1 - \tfrac{3}{8}x)$, or $= p\, f_s\, (1 - \tfrac{3}{8}x.)$ Then our equations for the solution of beams are as follows:

$$\tfrac{2}{3} f_c x = p f_s \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7).$$

$$x = -\tfrac{3}{4}\, ep + \sqrt{{}^{3}/_{2}\, ep\, (1 + \tfrac{3}{8}\, ep)} \quad \ldots\ldots\ldots\ldots\ldots (10).$$

$$M = K bd^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (15).$$

When definite values are assumed for e, p, f_s and f_c and inserted in the two coefficients $2/3\, f_c\, x\, (1 - \tfrac{3}{8}x)$ and $p\, f_s\, (1 - \tfrac{3}{8}x)$ two values will be found for K. The smaller of these values should be used for computing the resisting moment of the beam.

Values of K for different values of f_c, f_s, p and e may be worked out and tabulated. When this has been done, by selecting the proper value of K from the table and substituting it in equation (15) the solution of this equation for a given beam becomes very simple. Values of K for various values of p, e, f_s and f_c are given in Tables LXIII, LXIV, LXV and LXVI.

TABLE LXIII.

VALUES OF K, FOR VARIOUS RATIOS, p, OF STEEL.

d = 1. f_s = 12,000. f_c = 400.

	e=6.	e=7.5.	e=10.	e=12.	e=15.	e=20.	
p	K	K	K	K	K	K	p
.001	12	12	12	11	11	11	.001
.002	23	23	23	22	22	22	.002
.003	34	34	33	33	33	33	.003
.004	43	45	44	44	43	43	.004
.005	47	52	55	54	54	53	.005
.006	51	56	62	64	63	63	.006
.007	54	59	66	70	74	73	.007
.008	57	62	69	74	80	82	.008
.009	60	65	72	77	83	91	.009
.010	62	68	75	80	86	94	.010
.012	67	72	80	84	88	99	.012
.014	70	76	84	89	93	104	.014
.016	74	80	87	93	99	107	.016
.018	76	83	91	96	102	110	.018
.020	80	86	94	99	106	113	.020
.025	86	92	101	105	112	120	.025
.030	91	97	105	111	116	124	.030
.035	95	102	110	114	121	128	.035
.040	99	105	113	118	124	131	.040
.045	102	109	117	122	127	134	.045
.050	106	112	119	124	130	136	.050

TABLE LXIV.

VALUES OF K, FOR VARIOUS RATIOS, p, OF STEEL.

d = 1. f_s = 12,000. f_c = 500.

p.	e=6.	e=7.5.	e=10.	e=12.	e=15.	e=20.	p.
.001	12	12	12	11	11	11	.001
.002	23	23	23	22	22	22	.002
.003	34	34	33	33	33	33	.003
.004	45	45	44	44	43	43	.004
.005	56	55	55	54	54	53	.005
.006	64	66	65	64	64	63	.006
.007	68	74	75	75	74	72	.007
.008	72	78	86	85	84	82	.008
.009	75	81	91	95	93	92	.009
.010	78	85	93	100	103	101	.010
.012	83	90	100	105	110	120	.012
.014	91	95	105	111	116	124	.014
.016	93	100	109	116	124	130	.016
.018	95	104	114	120	128	133	.018
.020	100	107	118	123	132	138	.020
.025	107	115	123	131	140	142	.025
.030	114	121	131	138	145	149	.030
.035	119	127	137	143	151	155	.035
.040	124	131	142	148	155	160	.040
.045	128	136	145	152	159	164	.045
.050	132	139	149	155	162	176	.050

TABLE LXV.

VALUES OF K, FOR VARIOUS RATIOS, p, OF STEEL.

$d = 1$. $f_s = 16,000$. $f_c = 500$.

p.	e=6.	e=7.5.	e=10.	e=12.	e=15.	e=20.	p.
.001	16	15	15	15	15	15	.001
.002	31	30	30	30	30	29	.002
.003	45	45	44	44	44	43	.003
.004	54	60	59	58	58	57	.004
.005	59	65	73	72	72	70	.005
.006	64	69	78	83	85	84	.006
.007	68	74	82	88	95	97	.007
.008	72	78	86	92	100	109	.008
.009	75	81	90	96	104	114	.009
.010	78	84	94	100	107	118	.010
.012	83	90	100	106	110	124	.012
.014	88	96	105	112	116	130	.014
.016	93	100	110	116	124	133	.016
.018	96	104	114	120	128	138	.018
.020	100	107	118	124	133	142	.020
.025	107	114	125	132	140	149	.025
.030	114	122	132	138	146	155	.030
.035	119	127	138	142	151	160	.035
.040	124	132	142	148	155	164	.040
.045	128	136	146	152	160	170	.045
.050	131	139	150	156	162	176	.050

TABLE LXVI.

VALUES OF K, FOR VARIOUS RATIOS, p, OF STEEL.

$d = 1$. $f_s = 20,000$. $f_c = 600$.

	e=6.	e=7.5.	e=10.	e=12.	e=15.	e=20.	
p	K	K	K	K	K	K	p
.001	19	19	19	19	19	19	.001
.002	38	38	38	37	37	37	.002
.003	57	56	56	55	55	54	.003
.004	65	71	74	73	72	71	.004
.005	71	78	87	90	89	88	.005
.006	76	83	94	100	106	105	.006
.007	81	89	99	106	114	121	.007
.008	86	93	104	111	120	132	.008
.009	90	97	109	117	125	136	.009
.010	93	101	112	120	129	141	.010
.012	100	108	120	127	132	149	.012
.014	106	114	126	134	139	155	.014
.016	111	120	131	139	149	160	.016
.018	114	125	137	144	153	165	.018
.020	120	129	141	149	159	170	.020
.025	129	137	151	154	168	179	.025
.030	137	146	158	166	175	186	.030
.035	143	152	164	172	181	192	.035
.040	149	158	170	177	186	197	.040
.045	154	163	175	182	191	200	.045
.050	158	167	179	186	195	204	.050

The method of determining K and the manner of using it for determining the section of a beam will be shown by the solution of a problem.

Example.—Design a beam of 12.5 ft. span to carry at the age of two months a load of 1,000 lbs. per lin. ft., using a 1 : 2 : 4 Portland cement concrete, with a factor of safety of 5, and steel having an elastic limit of 40,000 lbs. per sq. in., with a factor of safety of $2\frac{1}{2}$ reckoned from the elastic limit.

From page 192 we find that the crushing strength of a 1 : 2 : 4 concrete at 1 month is 2,400 lbs. per sq. in. This gives us a working value of 480 lbs.; we will use 500 lbs. From Thacher's formula, page 209, we find the coefficient of elasticity of the concrete is about 3,000,000 lbs. per sq. in. The working value of the steel using a factor of safety of $2\frac{1}{2}$ at the elastic limit, is 16,000 lbs., and its coefficient of elasticity may be taken at 30,000,000.

Then
$$e = \frac{E_s}{E_c} = 10.$$

Inserting the values $e = 10$ and $p = .01$ in equation (10), it becomes

$$x = -\frac{3 \times 10 \times .01}{4} + \sqrt{\frac{3 \times 10 \times .01}{2}\left(1 + \frac{3}{8} \times 10 \times .01\right)}$$

solving

$$= -.075 + 0.3945,$$

or,

$$x = 0.3195 = 0.32, \text{ approximately.}$$

Now inserting the values $x = 0.32$ and $f_c = 500$ in formula (13),

$$K = \tfrac{2}{3} f_c x \left(1 - \tfrac{3}{8} x\right),$$

$$= \frac{2 \times 500 \times .32}{3}\left(1 - \tfrac{3}{8} \times .32\right),$$

and $K = 94$.

This value of K should be employed for, substituting the values $x = 0.32$, $p = .01$, and $f_s = 16{,}000$ in equation (14).

$$K = p f_s \left(1 - \tfrac{3}{8} x\right),$$

we find

$$K = 141.$$

Therefore

$$M = 94\ bd^2, \text{ and } bd^2 = \frac{M}{94}. \quad M = \tfrac{1}{8} W l,$$

$$= \frac{12{,}500 \times 150}{8} = 234{,}375 \text{ in. lbs.}$$

$$bd^2 = \frac{234,375}{94}$$

$$bd^2 = 2,494.$$

Assuming a width of beam,

$$b = 12 \text{ ins.}$$

$$d^2 = \frac{2,494}{12} = 208, \text{ approximately.}$$

$$d = \sqrt{208} = 14.4 \text{ ins.}$$

Using $14\frac{1}{2}$ ins. and adding $1\frac{1}{2}$ ins. of concrete to cover reinforcement, we have a total depth of beam of 16 ins.

$$A_c = b\,d = 14.5 \times 12 = 174 \text{ sq. ins.}$$

But the percentage of reinforcement equals 1 or .01, and $A_s = .01\ A_c$. $A_s = 1.74$ sq. ins. Three rods $\frac{7}{8}$ in. diameter give 1.804 sq. ins. area. Therefore, three $\frac{7}{8}$ in. diameter rods will be chosen for the reinforcement, and we have a 12×16-in. section reinforced with three rods $\frac{7}{8}$ in. in diameter.

Shearing Stresses in Reinforced Concrete Beams.—There is some diversity of opinion as to the action of the internal stresses in a reinforced concrete beam under flexure and the best manner to care for them. The nature of these stresses and the different systems devised to care for them, have already been explained, and we will now discuss the manner of determining whether reinforcement in the vertical plane of the beam is needed, the proper amount to be used and the most desirable location of the stirrups.

As we have seen, the shearing strength of concrete is somewhat in excess of its tensile strength. This strength will be found sufficient to care for all dangerous stresses due to vertical and horizontal shear in a majority of cases, with a normal amount of reinforcement for moderate and long spans. In comparatively short spans or deep girders heavily loaded, failure may be caused by shearing or by diagonal tension. Failure usually takes place at or near the quarter points or between these points and the ends by means of diagonal cracks slanting upward toward the center of the beam. These failures are said to be due partly to shear and partly to tension, or, more strictly speaking, they may be considered as due to tensile stresses induced by shear. The diagonal cracking is supposed to be due either to the slipping of the rod or to the rupture of the concrete by diagonal tension. With plain rods this may be due to a reduction of the section of the rod on account of the stretching of the metal. If the concreting is properly done, the adhesion should not fail in a beam of normal dimensions. Failure, due to stretching, can only take place when the stresses in the rods are at or near their elastic limit, and in that event the beams would soon fail from other causes.

Another and more common form of failure is by horizontal shearing at or slightly above the plane of the bottom or tension reinforcement. (If the stresses in the concrete along this plane be kept below a certain safe limit, longitudinal shearing will not take place. This may be accomplished by avoiding the use of too large rods, and where several rods are used, if a sufficient amount of concrete is kept between the rods, failure from this cause will be avoided.) The usual rule is to space the rods so that the distance between them is equal to or greater than 1½ times the diameter of the rod.

Another method of failure, sometimes called a shearing failure, is by diagonal tension in the concrete. This method of failure has already been explained.

The tendency toward shearing may be prevented by anchoring the rods at their ends. Provision is also made to care for both kinds of shearing and diagonal tension by bending up alternate rods at about the third points of the beams and running them on a slant such that they reach the upper portion of the beam near its end.

The Hennebique system employs these bent shearing rods, together with stirrups. Stirrups are used to prevent horizontal shearing, both with and without the bent rods.

The action of the internal stresses may be best understood by comparing the concrete beam with a metal plate girder. In the case of a single reinforcement the concrete replaces the web and compression flange, while the steel carries the entire tensile stress. The web of the beam cannot fail by buckling as in the plate girder, but may by shearing as above explained.

The horizontal shear at the plane of the reinforcement at any point may be determined by well known methods of analysis used in determining the flange stress in plate girders. Thus, if two points in the beam A and B be chosen at x distance apart, the moment and flange stress at these two points computed, the difference in stress between them will be the increment of stress or ɔrizontal shearing between them. This stress must be cared for y the shearing strength of the concrete supplemented, if need be ɔy some of the well known forms of stirrups. If the increment ɔf stress over the distance x, divided by the horizontal section of the concrete through which it is distributed, does not exceed the allowable shearing stress, no web reinforcing will be needed. If it is exceeded, a proper allowance may be made for the strength of the concrete, and metal of sufficient section provided to care for the remaining transverse shear.

Treating this shear from another standpoint, we have from well known principles of mechanics the following axiom, "that at every point in the beam the intensities of the vertical and horizontal shears are equal."* The resolution of these two shearing stresses at the neutral axis gives two equal and opposite stresses at right angles to each other—one compression and the other tension—making an angle of 45° with the neutral axis. At some distance from the axis these stresses, which are called secondary stresses, combine with the principal stresses in the top and bottom of the beam, giving the resulting lines of stresses as shown in Fig. 67. Theoretically, these stresses are best cared for by a tension reinforcement inclined upwards toward the ends at an angle of 45° to the axis of the beam. This is the principle followed in systems heretofore described having inclined stirrups. For the successful transmission of the increment of stress from the tension reinforcement to the concrete, which acts as the web and compression flange, it is necessary that the stirrups be firmly anchored to the longitudinal reinforcement. This is only obtained by the use of two or three patent systems, one of which, the Kahn system, has the stirrups sheared from the sides of the bar and forming an integral part of it. Nearly as good results may be secured with a V-stirrup hooked over the corrugated bar and in a less degree with other deformed bars. When thus used it is assumed that the combination causes a truss action analogous to the Pratt truss, the concrete being strained alone in compression, while the steel cares for all tensile strains. However, in the one case the use of patent bars is expensive, and the placing of concrete with stirrups inclined at an angle of 45° is difficult, and makes the proper tamping of the layers of concrete almost impossible. This is even more difficult when the stirrups are detached.

*Hiroi's Plate Girder Construction, p. 16.

A more rational arrangement would seem to be to use vertical stirrups to care for the vertical shear, securing thereby a truss action analogous to that of the Howe truss. This arrangement gives a more economical distribution of the metal, is just as efficient as either attached stirrups or stirrups used with deformed bars, without the additional cost of patented bars, and, lastly, concrete may be placed and tamped with much greater facility and at a lower cost. The author knows of no concrete girders thus reinforced with a proper amount of metal that have failed by shearing.

Some designers space the stirrups by empirical rule. Mr. E. L. Ransome's rule is to place the first stirrup a distance from the end of the beam corresponding to one-quarter of its depth, the second a distance of one-half its depth beyond the first, the third a distance of three-quarters the depth beyond the second, and the fourth a distance of the depth of the beam beyond the third. Other empirical rules are used by other designers, while several more or less theoretical formulas have been devised which need not be repeated in this place.

The size and location of the stirrups may be calculated as in a plate girder, treating the stress as actual shear. This is best done by drawing the shear diagrams for concentrated and distributed loads, it being assumed, as outlined above, that the beam acts as a Howe truss with diagonal compression members inclined at an angle of 45°. The resultants of these diagonal stresses are equal vertical and horizontal forces. It is assumed that a part of the horizontal forces is provided for by adhesion of the concrete to half of the surface of the tension members and the remainder resisted by the transverse shearing strength of the vertical rods. It has been found that several small rods give better results than one large one, as there is a more uniform distribution of stress. The rods are usually made of the same size and spaced closer together toward the ends, where the shearing stresses are higher. To determine the spacing an area equal to the adhesion is subtracted from the shear diagram and the remaining area is divided into panels such that each has an area equal to the maximum shear allowed for one rod or series of rods. As the height of the panels decreases their length increases giving a series of spaces representing graphically the spacing of each rod or series of rods. This will be understood by referring to Fig. 180, which represents the shear diagram of a beam with a uniform and concentrated loading. The area above the loading A B represents the shear due to a concentrated load P, and that below the line the shear, due to uniformly distributed load, considering only the portion of the shear diagram to the right of

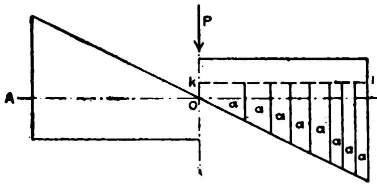


Fig. 180.—Shear Diagram of Beam with Uniform and Concentrated Loading.

the center line of the beam. Then if the area above the dotted line *k-l* represents the allowable stress cared for by the adhesion of the rods, the portion of shear in the diagram below this line must be provided for by stirrups. If this be divided into equal areas *a*, such that the amount of shear represented by area *a* will be cared for by each stirrup, the resulting linear dimensions of the trapezoids *a* give graphically the desired stirrup spacing.

T-Beams.—T-beams are extensively used in floor construction, and with considerable economy of materials. A portion of the floor slab connecting the ribs or girders is considered as acting as flange area. This may be safely done when the girder or rib is built as a monolith with the floor slab. In many cases this is not done; under such conditions the girder should be designed with a sufficient area of concrete to develop the full strength of the reinforcing metal in the tension flange. There is considerable difference of opinion among engineers as to what portion of the width of the floor slab may be considered as furnishing flange area for the T-beam. It is usual to assume that a width equal to a certain number of times the width of the stem of the T, usually from 3 to 10 times, should be used. It is evident that, under normal conditions, ten times the width of the stem "b" of the rib is far too liberal an assumption, while a value of three is much too small; a value of from four to six is probably about right to fit all cases without straining the concrete too high.

It will be readily understood that if the construction is monolithic, the slab will be fixed at the beam, and when loaded the concrete will be strained in compression in the under side of the slab out to the point of contraflexure at about the quarter points of the slab span. If the concrete is strained at right angles to the direction of the floor span due to the slab acting as flange to a T-beam, we see that the concrete will be strained in two directions, or will have a stress double that usually assumed for the unit-working stress. If both primary and secondary rib systems are used, a portion of the concrete will be strained to nearly three time's the assumed unit-working stress.

On account of the fixing of the slab over the beam, some engineers assume that the concrete may be considered as furnishing flange area out to the point of contraflexure, giving a width of one-half the distance center to center of beams. Others only allow one-third the distance center to center of beam.

A more rational method would be to take the width B of the

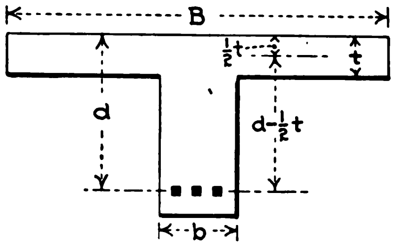


Fig. 181.—T-Beam Diagram.

slab used for flange area as a certain number of times the thickness t of the slab. For:

Let S_h represent the total shear between rib and flange along their plane of union.

Let S_v represent the total shear in the flange along vertical planes that are a continuation of the planes forming the sides of the rib, and which cut off the wings of the flange. Then for equal strength in shear t should equal $\frac{1}{2}$ b. When b is the width of the rib, b should never be less than 2 t, although t may be less than $\frac{1}{2}$ b. Then the flange width B may be taken as 10 t, which, if 2 t = b, will be equal to 5 b.

Referring to Fig. 181, the lever arm between the compression and tension forces is d — $\frac{1}{2}$ t. If B is taken equal to 10 t, the compression area will be 10 t^2. The stresses in the slab, however, will vary from a maximum at the rib to 0 at the outer edges of the flange B. These compression flange stresses may be assumed to vary as the ordinates of a parabolic segment, and the area available for compression will be $^2/_3 \times 10\ t^2$, and the total compressive stress will be $^{20}/_3\ f_c\ t^2$. This will be on the side of safety, as a portion of the stem of the T is generally available for flange area. Again, the center of gravity of the parabola is either $5/8$ t or $^3/_5$ t, above the lower face of the slab, depending upon whether the origin of the parabola is taken at the top or bottom of the slab. Thus the assumption that the center of gravity is at $1/2$ t above the bottom face of the slab is on the side of safety.

Formulas 7, 10 and 15, given above for rectangular beams, may be used without difficulty for designing T-beams. They will hold true if the neutral axis coincides with or is above the under side of the slab; if it falls below the under side of the slab the formula will err on the side of safety. The position of the neutral axis will, of course, depend upon the values of B, b, t, d, and the amount of metal used.

An empirical method of design which, with slight modifications, has been extensively used, is as follows: Assume that the floor slab furnishes the necessary compression flange area, and assume its center of gravity at the center of the flange thickness. In order that sufficient area of concrete shall be available to develop the full strength of the steel, the area $\frac{20}{3}\ t^2$ should not be less than $\frac{f_s}{f_c} A_s$.

Thus if $f_s = 16{,}000$, and $f_c = 500$, $\frac{20\ t^2}{3}$ should not be less than $32\ A_s$.

If the bending moment M and width B are known, and thickness t and amount p of steel assumed, d may be approximated from the formula $M = K\ bd^2$, taking $bd = {}^{20}/_3\ t^2$, and taking the value of K from Table XLIV. Then

$$d = \frac{M}{\frac{20}{3} K t^2}$$

When the depth d is fixed, and it is desired to obtain A_s, the area of the steel, $M = (d - 1/2\ t)\ f_s\ A_s$, or

$$A_s = \frac{M}{f_s (d - \frac{1}{2} t)}.$$

It should be remembered, however, that $\frac{20 t^2}{3}$ should not be less than $32 A_s$, and b should not be less than 2 t. It is desirable that stirrups be used to connect the floor slab to the stem of the T.

A number of theoretical formulas have been developed by different engineers for the solution of T-beams. No one of them, however, is entirely satisfactory, as they are all based upon more or less doubtful assumptions. Again, the mathematical work necessary to their solution is quite complicated, especially as it is usually necessary to determine the position of the neutral axis, at best a tedious solution. When it is desired to use such a formula, either for designing new work or for checking work designed by some such empirical formula as that given above the formula developed by Capt. John S. Sewell, given on page 325 should be used. It should, however, be noted that the origin of the parabola is taken at the neutral axis instead of at the extreme fibre, as is usually done. This changes slightly the position of the center of gravity of the compression area, it being $^3/_5$ d instead of ⅝ d above the neutral axis. This is on the side of safety.

It is to be regretted that few tests have been made on T-beams, and these tests were made to destruction, giving no data of value for the development of formulas for the solution of T-beams. It is to be hoped that tests will be undertaken and suitable data obtained in the near future for the development of rational T-beam formulas.

Beams with Double Reinforcements.—When excessive loads are to be carried and it is unadvisable to increase the depth of the beam the compression flange is sometimes strengthened by the addition of reinforcing metal. In order that the concrete and the steel in the compression flange act together as a homogeneous material, it is necessary that the stresses carried by the concrete and the metal shall be proportional to their respective coefficients of elasticity, i. e.,

$$\frac{f_s^c}{f_c} = \frac{E_s}{E_c}.$$

On account of this law, quite low unit stresses will result in the steel in compression, and the large amount of metal needed for the compression flange will not give an economic form of construction. In fact, some authorities hold that double reinforcements are never an economic form of construction, although they have been extensively used by some European engineers.

To determine the effect of double reinforcement, let us first assume a beam of rectangular cross-section, with sufficient area of reinforcement A_s in its tension flange to develop the proper working stresses in the concrete of the compression flange. Let d, in Fig. 182, be the effective depth of the beam, and xd the distance of the neutral axis below the top of the beam.

If a compression reinforcement having an area A^c_s be added to the compression flange at a distance z above the neutral axis, the

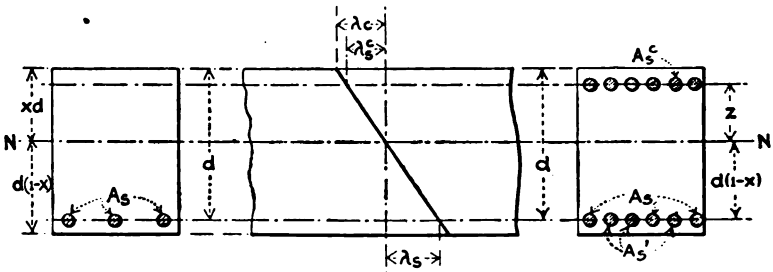


Fig. 182.—Stress Strain Diagram of Beam with Double Reinforcement.

position of the neutral axis will be changed if the beam is loaded as before. If, however, when the compression reinforcement A^c_s is placed in the compression flange, additional steel having an area A'_s is added in the tension flange to develop the strength of the compression reinforcement A^c_s, the position of the neutral axis will remain unchanged. In order that this may obtain we must have the relation

$$\frac{A_s'}{A_s^c} = \frac{z}{d(1-x)} \quad \ldots\ldots\ldots\ldots\ldots (16)$$

But in order that the steel shall undergo the same deformation as the concrete, we must have the relation

$$\frac{f_s}{f_s^c} = \frac{d(1-x)}{z} \quad \ldots\ldots\ldots\ldots\ldots (17),$$

Multiplying by equation (16), we obtain

$$\frac{f_s A_s'}{f_s^c A_s^c} = 1, \text{ or } f_s A_s' = f_s^c A_s^c \quad \ldots\ldots\ldots\ldots\ldots (18),$$

that is, equal forces have been added to each side of the beam. The resisting moment M^1 added to the beam will be equal to

$$M^1 = f_s A_s' \left[z + d\ (1 - x)\right] \text{ in inch pounds.}$$

But

$$\frac{f_s^c}{f_c} = \frac{E_s}{E_c} = e, \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (19),$$

or,

$$(f_s^c = e\, f_c),$$

and

$$f_s A_s' = e\, f_c A_s^c,$$

or,

$$A_s' = \frac{e\, f_c A_s^c}{f_s}.$$

The resisting moment of a beam with a single reinforcement is from equation (11).

$$M = {}^5/_{12}\, f_c b x^2 d^2 + A_s f_s d\ (1 - x),$$

and for the resisting moment of a beam with double reinforcement, we will add the quantity obtained in equation (18), and we obtain

$$M = {}^5/_{12}\, f_c b x^2 d^2 + A_s f_s d\ (1 - x) + f_s A_s' \left[z + d\ (1 - x)\right] \quad \ldots. (20),$$

remembering that

$$A_s' = \frac{e\, f_c A_s^c}{f_s}.$$

To determine the effect of double reinforcement, we will take the beam designed on page 307 and note the effect of doubling the reinforcement, and then add enough steel to the compression flange to balance the increase of steel in the tension flange and note the increase in strength obtained thereby.

From page 307 we have the following data: Span = 12.5 ft., resisting moment = 234,375 in. lbs., b = 12 ins., d = 14.5 ins., p. = .01, x = .32 d = 4.65 ins., d (1 — x) = 9.85 ins., A_s = 1.74 sq. ins., net, or using three rods ⅞ ins. in diameter = 1.804 sq. ins.

Doubling the reinforcement A_s = 3.60 sq. ins. of metal. It is not necessary in this case to determine the value of x as K may be taken directly from Table LXV., K = 118. From equation (15) $M = K\, bd^2$, hence M = 297,714 in. lbs., which is the resisting moment obtained by the doubling the area of reinforcement, or a total gain of 63,339 in. lbs.

Now let us place enough steel in the compression flange to care for the added 1 per cent. = 1.8 sq. ins. of steel, $A'_s = 1.8$.

Now from equation (19),

$$A_s' = \frac{e\, f_c\, A_s^c}{f_s}.$$

$$f_c = 500, \qquad f_s = 16{,}000, \qquad e = 10,$$

and

$$A_s^c = \frac{1.8 \times 16{,}000}{10 \times 500} = 5.76 \text{ sq. ins.}$$

Six rods $1\frac{1}{8}$ ins. diameter = 5.94 sq. ins., will be used, placing them 1.5 ins. below the top face of the beam and the distance between the steel in tension and compression, i. e., the lever arm of forces added is 13 ins., which equals $z + d\ (1 - x)$ in equation 18. The resisting moment obtained by the use of the steel in compression is

$$16{,}000 \times 1.8 \times 13 = 374{,}400 \text{ in. lbs.}$$

it being remembered that 1.8 sq. in. of steel in tension is balanced by 5.76 sq. ins. of steel in compression. The total resistance of our beam now is

$$234{,}375 + 374{,}400 = 608{,}775 \text{ in. lbs.}$$

The resisting moment obtained by the use of 2 per cent. of reinforcement was 297,714 in. lbs., and we see that by the addition of enough steel to balance 1 per cent. of steel in tension, the resisting moment is more than doubled, and is 2.6 that when only 1 per cent. of steel was used, or considering it from another standpoint, the addition of 4.34 per cent. of steel increases the strength of the beam 2.6 times.

The excessive amount of steel necessary for the compression flange, 1.67 times that used in tension, should be noted. This is due, as has been stated, to the necessity of having the same deformation in the steel and concrete. When a lower percentage is used than that necessary to fulfill this requirement, the internal stresses will not be in accordance with the usual accepted and safe assumptions.

CHAPTER XX.

VARIOUS BEAM THEORIES.

Formula for Beams, Based on a Rectilinear Distribution of Stress.—We will assume a rectangular beam under flexure. Using the same nomenclature as heretofore and neglecting the tensile stress of the concrete, the elastic deformations of the concrete above the neutral axis will be represented graphically by the triangle a o b in Fig. 183 (b), following the theory of conservation of sectional planes. The triangle A o B, Fig. 183 (c), will represent the total compressive stress on a unit width of concrete, f_c being the maximum intensity of stress, assuming that

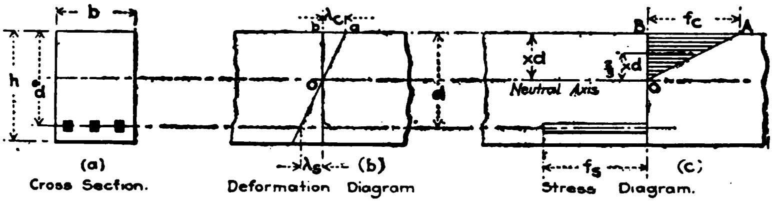


Fig. 183.—Stress Strain Diagram of Beam Assuming Rectilinear Distribution of Stress.

the modulus of elasticity of the concrete is constant throughout the whole range of stress. This assumption gives a rectilinear relation between the stress and strain in the concrete.

The total stress on the concrete for a unit width of beam then equals $\frac{1}{2} f_c xd$, and the total compression on the section of a beam having a width b is

$$F_c = \tfrac{1}{2} f_c xd b.$$

The total tension on the steel is

$$F_s = A_s f_s \qquad (21),$$

These two forces are equal, and

$$A_s f_s = \tfrac{1}{2} f_c xd b \qquad (22),$$

Reducing, remembering that $\dfrac{A_s}{A_c} = p$, and $A_c = bd$,

$$p f_s = \tfrac{1}{2} f_c x \qquad (23),$$

But the relations between λ_c, λ_s, f_c, f_s, E_c and E_s are the same as those explained on page 302, and are expressed by eq. (3).

$$f_c = \frac{E_c x f_s}{E_s (1 - x)} \qquad (3).$$

...ostituting this value in eq. (23), we ...

$$p f_s = \frac{1}{2} \frac{E_c}{E_s} \frac{x^2 f_s}{(1 - x)},$$

...ucing

$$\frac{1}{2} x^2 = \frac{E_s}{E_c} (1 - x) p \quad \ldots\ldots (24),$$

rom which

$$x = -\frac{E_s}{E_c} p \pm \sqrt{2 \frac{E_s}{E_c} p\left(1 + \frac{1}{2} \frac{E_s}{E_c} p\right)} \quad \ldots\ldots (25),$$

or replacing $\frac{E_s}{E_c}$ by e,

$$x = -ep \pm \sqrt{2 ep (1 + \frac{1}{2} ep)} \quad \ldots\ldots (26),$$

The resisting moment of the beam may now be determined as equation (26) gives the position of the neutral axis. Taking the center of moments about the neutral axis the total resisting moment will be equal to the sum of the moments of compression in the concrete and the moment of tension in the steel, and we have

$$M = \frac{1}{2} f_c x d b \times \frac{2}{3} x d + A_s f_s d (1 - x) \quad \ldots\ldots (27),$$
$$= \frac{1}{3} f_c b x^2 d^2 + A_s f_s d (1 - x),$$

reducing as before.

$$M = [\frac{1}{3} f_c x^2 + p f_s (1 - x)] b d^2 \quad \ldots\ldots (28),$$

Equation (28) contains both f_s and f_c, and to reduce it to a convenient form for use, we will substitute successively the values of f_s and f_c from equation (23). Substituting and reducing, equation (28) takes the form

$$M = \frac{1}{2} f_c x (1 - \frac{1}{3} x) b d^2 \quad \ldots\ldots (29),$$

when the allowable stress for the concrete is assumed, and the form

$$M = p f_s (1 - \frac{1}{3} x) b d^2 \quad \ldots\ldots (30),$$

when the allowable stress in the steel is assumed.

As before the coefficient $p f_s (1 - 1/3 x)$ will be found to be the determining factor when a low percentage of steel is used o... when a moderately low percentage of steel with a concrete c... high strength is used.

The coefficient $\frac{1}{2} f_c x (1 - \frac{1}{3} x)$ will be the determining fa... tor when a high percentage of steel is used, when a steel of hi... elastic limit is used and when concrete of low crushing stren...

... these coefficients are obtained as expl...

in Chapter XIX., in connection with the discussion of beams under a parabolic distribution of stress, which may be represented by a constant K and, as before, we have for the solution of beams the equations

$$p f_s = \tfrac{1}{2} f_c x \qquad (23),$$

$$x = -ep \pm \sqrt{2 ep (1 + \tfrac{1}{2} ep)} \qquad (26),$$

and

$$M = K bd^2 \qquad (31),$$

in which the coefficient K of equation (31) is obtained from the coefficients

$$K = p f_s \ (1 - \tfrac{1}{3} x),$$

or

$$= \tfrac{1}{2} f_c x \ (1 - \tfrac{1}{3} x).$$

The smaller value obtained from these coefficients should be used. If desired a number of values of K may be obtained by using various values of p, e, f_s and f_c and tabulated, then by selecting the proper value for use under the given conditions, the solution of the given problem will be greatly simplified.

Example.—Design a beam using Formulas (23), (26), (31), having given the same data as was used for the example on page 307. From page 307 we have the following data: M = 234,375 lbs., e = 10, p = 01, f_c = 500 and f_s = 16,000.

From equation (26) we have

$$x = -10 \times .01 + \sqrt{2 \times 10 \times .01 \ (1 + \tfrac{1}{2} \times 10 \times .01)},$$

from which

$$x = 0.358.$$

To determine value of K, we insert value of f_s = 16,000 lbs. in coefficient $K = p f_s \ (1 - \tfrac{1}{3} x)$,

and

$$K = .01 \times 16{,}000 \ (1 - \frac{.358}{3}),$$

from which

$$K = 121.$$

Again inserting value f_c = 500 in coefficient.

$$K = \tfrac{1}{2} f_c x \ (1 - \tfrac{1}{3} x).$$

$$K = \tfrac{1}{2} \times 500 \times .358 \ (1 - \tfrac{1}{3} \times .358),$$

or

$$K = 79.$$

Hence the proper value of K to be used is 79: and we have

$$M = 79 \ bd^2.$$

but M = 234,375, and we will assume b = 12. We then have

$$234{,}375 = 79 \times 12 \ d^2,$$

from which

$$d^2 = 247.2,$$

and

$$d = 15.72 \text{ ins. depth of beam required.}$$

We will use 16 ins. and the area of beam = 12 × 16 = 192 sq. ins.; 1 per cent. of this is 1.92 sq. ins., which is amount of reinforcement required. Three rods $^{13}/_{16} \times {}^{13}/_{16}$ ins. = 1.98 sq. ins. Adding 1.5 ins. of concrete below the reinforcement we obtain a beam 12 × 17½ ins., reinforced with three $^{13}/_{16} \times {}^{13}/_{16}$-in. rods.

Formulas for Beams, Based on Distribution of Stress, Proposed by Capt. John S. Sewell.—We will assume as before a rectangular beam under flexure. As explained on page 293, the elastic deformations above the neutral axis, as found by Capt. Sewell's study of Watertown Arsenal Tests on Concrete Cubes, under compression, are represented by a line lying between the curve representing a parabolic distribution and the straight line representing a rectilinear distribution of stress. This may be shown graphically by the curve o x t in Fig. 184, in which the curve

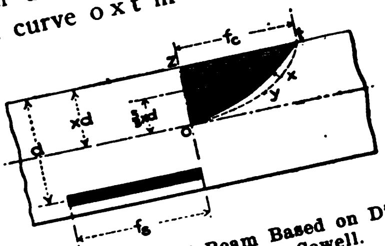


Fig. 184.—Stress-Strain Diagram of Beam Based on Distribution of Stress Assumed by Capt. J. S. Sewell.

o y t is the curve limiting the parabolic distribution and the line o t the rectilinear distribution of stress.

The height of the center of gravity of the area o x t z above the neutral axis, as was stated on page 294, may be taken at $^{3}/_{5}$ x d and the area of stress is taken at ⅝ f_c x d. The total compressive stress on the section of a beam having a width b is:

$$F_c = \tfrac{5}{8} f_c x d b \quad \ldots\ldots (32),$$

f_c being the maximum intensity of stress on the concrete.

As before, neglecting the tensile strength of the concrete, the total tensile stress carried by the steel is

$$F_s = A_s f_s \quad \ldots\ldots (5)$$

But these two forces are equal, and

$$A_s f_s = \tfrac{5}{8} f_c x d b \quad \ldots\ldots (3$$

Reducing, remembering that

$$\frac{A_s}{A_c} = p, \text{ and } A_c = bd,$$

$$p f_s = \tfrac{5}{8} f_c x \quad \ldots\ldots$$

But the same relations exist between λ_c, λ_s, f_c, f_s, E_c and E_s as was explained on page 302, and there exists the relation

$$f_c = \frac{E_c \times f_s}{E_s\ (1 - x)} \quad \ldots\ldots\ldots\ldots (3).$$

Substituting this value in equation (34), we obtain

$$\tfrac{5}{8}\, x^2 = \frac{E_s}{E_c}\, p\ (1 - x) \quad \ldots\ldots\ldots\ldots (35),$$

From this

$$x = -\,{}^4\!/_5\, \frac{E_s}{E_c}\, p \pm \sqrt{\frac{8}{5}\, \frac{E_s}{E_c}\, p \left(1 + \frac{2}{5}\, \frac{E_s}{E_c}\, p\right)} \quad \ldots\ldots (36),$$

or replacing $\frac{E_s}{E_c}$ by e.

$$x = -\,{}^4\!/_5\, e\, p \pm \sqrt{\frac{8}{5}\, e\, p \left(1 + \frac{2}{5}\, e\, p\right)} \quad \ldots\ldots\ldots\ldots (37),$$

The resisting moment of the beam may now be determined as eq. (37) gives the position of the neutral axis. Taking the center of moments about the neutral axis, the total resisting moment will be equal to the sum of the moments of compression in the concrete and tension in the steel, and we have:

$$M = \tfrac{5}{8}\, f_c\, x\, d\, b \times {}^3\!/_5\, x\, d + A_s\, f_s\, d\ (1 - x),$$

reducing

$$M = [\tfrac{3}{8}\, f_c\, x^2 + p\, f_s\ (1 - x)]\ bd^2 \quad \ldots\ldots\ldots\ldots (38),$$

But equation (38) may be reduced to a convenient working form, as was explained on page 304. By substituting successively the values of f_s and f_c from equation (34), we obtain

$$M = \tfrac{5}{8}\, f_c\, x\ (1 - \frac{2}{5}\, x)\ bd^2,$$

when the allowable stress for the concrete is assumed, and

$$M = p\, f_s\ (1 - {}^2\!/_5\, x)\ bd^2,$$

when the allowable stress in the steel is assumed.

As was explained before, the coefficient $p\, f_s\ (1 - {}^2\!/_5\, x)$ will be the determining factor when a low percentage of steel is used, or when a moderately low percentage of steel with a concrete of high strength is used. The coefficient $\tfrac{5}{8}\, f_c\, x\ (1 - {}^2\!/_5\, x)$ will be the determining factor when a high percentage of steel is used. when a steel of high elastic limit is used and when concrete of low crushing strength is used.

Definite values of these coefficients may be obtained and tabulated, as before explained.

Then, for the solution of a beam, using Capt. Sewell's distribution of stress, we have the equations:

$$p f_s = \tfrac{5}{8} f_c x \ldots\ldots\ldots\ldots (34),$$

$$x = - \tfrac{4}{5} ep \pm \sqrt{\frac{8}{5} ep \left(1 + \frac{2}{5} ep\right)} \ldots\ldots\ldots\ldots (37),$$

$$M = K\ bd^2 \ldots\ldots\ldots\ldots (38),$$

The value of K in equation (38) being determined from one of the relations,

$$K = \tfrac{5}{8} f_c x (1 - \tfrac{2}{5} x),$$

or

$$= p f_c x (1 - \tfrac{2}{5} x).$$

The solution of a beam is given in the following example:

Example.—Design a beam, using Formulas (34), (37) and (38), having given the same data as was used for the example on page 307. From page 307 we have the following data: $M = 234{,}375$, $e = 10$, $p = .01$, $f_c = 500$ and $f_s = 16{,}000$. From eq. 37 we have

$$x = - \tfrac{4}{5} \times 10 \times .01 \pm \sqrt{\frac{8}{5} \times 10 \times .01 \quad (1 + \frac{2}{5} \times 10 \times .01)}$$

from which

$$x = .328.$$

To determine the value of K we insert the values of x and $f_s = 16{,}000$ in the formula for the coefficient, which is $K = p f_s (1 - \tfrac{2}{5} x)$, and

$$K = .01 \times 16{,}000 (1 - \tfrac{2}{5} \times .328),$$
$$K = 139.$$

Again inserting value of $f_c = 500$ in the coefficient,

$$K = \tfrac{5}{8} f_c x \ (1 - \tfrac{2}{5} x).$$
$$K = \tfrac{5}{8} \times 500 \times .328 \ (1 - \tfrac{2}{5} \times .328).$$
$$K = 89.$$

Hence, using the smaller value of $K = 89$, eq. 38 becomes

$$M = 89\ bd^2.$$

But $M = 234{,}375$, and assuming $b = 12$ ins., we have

$$234{,}375 = 89 \times 12\ d^2,$$

from which

$$d^2 = 219.4,$$

and

$$d = 14.8 \text{ ins. depth of beam required.}$$

We will use 15 ins.

Area of beam $= 12 \times 15 = 180$ sq. ins., 1 per cent. of reinforcement is used, $180 \times .01 = 1.8$ sq. ins. $=$ area of steel required. Three rods ⅞-in. diameter give an area of 1.804 sq. ins., which will be ample. Adding 1.5 ins. of concrete below the reinforcement we obtain a 12 x 16½-in. beam, reinforced with three ⅞-in. diameter rods.

T-Beam Formula.—The following T-Beam formula* was developed by John S. Sewell, Capt. Corps of Engrs., U. S. A.

The assumptions are: Plane sections, unity of action of concrete and steel, no initial stresses due to setting strains, loads vertical, beam horizontal, steel takes all tensile stress. The stress-strain curve is a parabola, with vertex on neutral axis, as A, Fig. 185. Let the dimensions of the sections be denoted by the letters in the figure, all being expressed in inches. Assume that the center of gravity of the area, B C G H, is at the center of the flange thickness, so that its lever arm with respect to A is $y_s + \frac{t}{2}$.

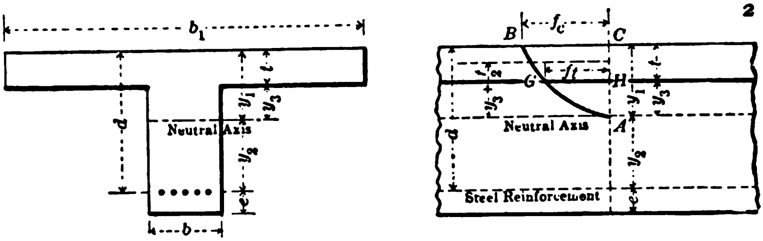


Fig. 185.—Diagram Illustrating T-Beam Theory of Capt. J. S. Sewell.

This is slightly in error, on the safe side. Assume that in any horizontal plane, the compression flange stresses vary as the ordinates of a parabolic segment, having its vertex where its plane cuts the curve A B, and its axis coincident with the line of intersection of its plane, and a longitudinal vertical plane through the middle of the rib; the maximum ordinate of the segment being at the vertex, and equal to the F ordinate of the curve A B at the same point; its ordinates at the edges of the flange reducing to 0

Let E_s represent the modulus of elasticity of steel.
E_c represent the modulus of elasticity of concrete.
T represent the unit tensile stress in the steel.
t_s represent the elastic limit of steel.
F represent the unit compressive stress in the concrete.
f_c represent the ultimate compressive strength of concrete per unit of area.
b_1 represent the width of flange of T.
b represent the width of the beam, in inches.
a represent the sectional area of steel per inch of width.

*Theory developed from that of Mr. Johnson, (see page 372).

represent the total sectional area of steel in the beam.
represent the distance of the neutral axis from extreme element in compression.
$_2$ represent the distance of the neutral axis from center of gravity of steel reinforcement.
λ_1 represent the compression in the extreme element of concrete, per unit of length.
λ_2 represent the elongation in the steel, per unit of length.
P_c represent the total compressive stress in the concrete.
P'_c represent the total compressive stress in the stem of the T.
P''_c represent the total compressive stress in the flange of the T.
P_s represent the total tensile stress in the steel.
L represent the length of the span, in feet.
s represent the shearing strength of concrete.
S_h represent the total shear between rib and flange, along their plane of union.
S_v represent the total shear in the flange along vertical planes cutting off the wings of the flange.

$$y_1 + y_2 = d \quad \ldots\ldots (1).$$

Then,
$$\frac{\lambda_2}{\lambda_1} = \frac{y_2}{y_1},\ \lambda_1 = \frac{F}{E_c},\ \text{and } \lambda_2 = \frac{T}{E_s}.$$

Therefore,
$$\frac{y_2}{y_1} = \frac{T\,E_c}{F\,E_s},\ \text{and } y_2 = \frac{T\,E_c}{F\,E_s}\,y_1 \quad \ldots\ldots (2).$$

Therefore,
$$y_2 = \frac{t_s\,E_c}{f_c\,E_s}\,y_1 \quad \ldots\ldots (2a).$$

From the assumptions made for T-beams, and the conditions of equilibrium,
$$P_c = P_c' + P''_c = P_s = a\,b\,T \quad \ldots\ldots (3).$$

Therefore
$$a\,b = \frac{P_c}{T} = \frac{P_s}{T} \quad \ldots\ldots (4).$$

or,
$$a\,b = \frac{P_c}{t_s} = \frac{P_s}{t_s} \quad \ldots\ldots (4a$$

From Johnson's Equation 6, in his "Materials of Construction," p.
$$s = \frac{f_c}{2\ \tan.\ \theta},$$

in which θ is the angle of rupture under direct compressive stress, usually about 60°, whence
$$s = \frac{f_c}{3.464};$$

but it is thought best to assume, for safety,
$$s = \frac{f_c}{8}.\quad P'_c = S_h = S_v$$

(not strictly true for S_v, as a part of P''_c is not transmitted to t the error is on the safe side).

$S_h = \frac{1}{2} s \times b \times \frac{1}{2} L \times 12 = 3\, b\, s\, L.$
$S_v = \frac{1}{2} s \times 2\, t \times \frac{1}{2} L \times 12 = 6\, t\, s\, L.$

It is evident that, for equal strength in shear, b should be at least twice as great as t, and t practically equal to ½ b.

The equation of the stress-strain curve, referred to A as the origin of co-ordinates, is,

$$F^2 = \frac{f_c^2}{y_1} y.$$

Hence,

$$f_t = \left(\frac{f_c^2}{y_1} y_3\right)^{\frac{1}{2}}.$$

Therefore,

$$P_c' = \tfrac{2}{3} y_3 \left(\frac{f_c^2}{y_1} y_3\right)^{\frac{1}{2}} b$$

$$= \tfrac{2}{3} b\, f_c\, y_1 \left(\frac{y_3}{y_1}\right)^{3/2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5).$$

From the equation of the stress-strain curve, and the properties of the horizontal parabola assumed as the curve of stress in the flange, we have

$$P_c'' = \tfrac{2}{3} b_1 \left[\tfrac{2}{3} f_c y_1 - \tfrac{2}{3} f_c y_1 \left(\frac{y_3}{y_1}\right)^{3/2}\right] =$$

$$\tfrac{4}{9} b_1 f_c y_1 \left[1 - \left(\frac{y_3}{y_1}\right)^{3/2}\right] \ldots\ldots\ldots\ldots\ldots\ldots\ldots (6).$$

$$S_h = 3\, b\, s\, L = P_c'' = \tfrac{4}{9} b_1 f_c y_1 \left[1 - \left(\frac{y_3}{y_1}\right)^{3/2}\right]$$

$S = \frac{f_c}{8}$, hence

$$\tfrac{3}{8} b\, f_c\, L = \tfrac{4}{9} b_1 f_c y_1 \left[1 - 1\left(\frac{y_3}{y_1}\right)^{3/2}\right]$$

Therefore,

$$b_1 = \tfrac{27}{32} \frac{b\, L}{y_1 \left[1 - \left(\frac{y_3}{y_1}\right)^{3/2}\right]} \ldots\ldots\ldots\ldots\ldots (7).$$

Substituting this value in Equation 6, we get,

$$P_c'' = \tfrac{3}{8} f_c\, b\, L \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8).$$

$$P_c = P_c' + P_c'' = \tfrac{3}{8} b\, f_c\, L + \tfrac{2}{3} b\, f_c\, y_1 \left(\frac{y_3}{y_1}\right)^{3/2}$$

$$= b\, f_c \left[\tfrac{3}{8} L + \tfrac{2}{3} y_1 \left(\frac{y_3}{y_1}\right)^{3/2}\right]$$

$$M = \tfrac{2}{5} y_3 P_c' + \left(y_3 + \frac{t}{2}\right) P''_c + P_s y_2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

The numbered equations are sufficient for designing.

Determine t from the requirements of the floor slab; assume d (if possible, so that d will be at least $= 4$ t). Compute P_c', P_c'' and P_s—all in terms of b. Substitute in equation 9, and b may be determined. If b $>$ 2 t, t must be increased; in designing T-beams, as in rectangular beams, compute M for a load 2½ times greater than the working load. Make $F = f_c$, and $T = t_s$.

It will be found that, for similar assumptions as to the relative values of d and t and the same values of f_c and E_c, designs of similar cross-section will be obtained for different loads and spans, so that the moment of the stresses may be written, quite accurately, in the form, $M = k f_c t b_1 \left(y_2 + y_3 + \frac{t}{2}\right)$

It will also be found that b_1 will be practically equal to b, multiplied by a constant. This enables a design of a certain type to be detailed quite simply and quickly, and also gives a means of quickly determining the approximate stresses in a given design, after the constant k and the ratio of b_1 to b have been determined for various types of designs, i. e., for various groups of assumptions as to the values of f_c and E_c combined, with the various values of t, corresponding to different spans and loadings.

Prof. A. N. Talbot's Beam Formulas.—The following theory of beams developed by Prof. Arthur N. Talbot deserves careful consideration, as it is based on data obtained from the latest tests on reinforced concrete beams. Unusual care was taken in conducting these tests at the University of Illinois, and it is believed that the data obtained therefrom are by far the most reliable now available.

The usual assumptions that the loads are applied at right angles to the length of the beam, that the supports will permit free longitudinal movement, that a plane section before bending remains a plane section after bending, and that the metal and surrounding concrete stretch together, are made. It is further assumed that the tensile strength of the concrete is negligible in the part of the beam where the bending moment is greatest, at least in the calculation of the resisting moment of the beam at the time of maximum load. The analysis is restricted to rectangular beams with reinforcement on the tension side only, and refers generally to simple beams free from end restraints.

Notation.—The following notation will be used:

b = breadth of rectangular beam.
d = distance from the compression face to the center of the metal reinforcement.
A = area of cross section of metal reinforcement.
$p = \frac{A}{bd}$ = ratio of area of metal reinforcement to area of concrete above center of reinforcement.
o = circumference or periphery of one reinforcing bar.
m = number of reinforcing bars.
E_s = modulus of elasticity of steel.
E_c = initial modulus of elasticity of concrete in compression, a term which will be defined.
$e = \frac{E_s}{E_c}$ = ratio of two moduli.
f = tensile stress per unit of area in metal reinforcement.
c = compressive stress per unit of area in most remote fiber of concrete.
c' = compressive stress per unit of area which causes failure by crushing.
λ_s = deformation per unit of length in the metal reinforcement.
λ_c = deformation per unit of length in most remote fiber of the concrete.
λ_c' = deformation per unit of length when crushing failure occurs; i. e., ultimate or crushing deformation.
$q = \frac{\lambda_c}{\lambda_c'}$ = ratio of deformation existing in most remote fiber to ultimate or crushing deformation.
k = ratio of distance between compression face and neutral axis to distance d.
z = distance from compression face to center of gravity of compressive stresses.
d' = distance from the center of the reinforcement to center of gravity of compressive stresses.
ΣX = summation of horizontal compressive stresses.
M = resisting moment at the given section.
s = horizontal tensile stress per unit of area in the concrete.
t = diagonal tensile stress per unit of area in the concrete.
u = bond stress per unit of area on the surface of the reinforcing bars.
v = vertical shearing stress and horizontal shearing stress per unit of area in the concrete.

Relation between Stress and Deformation for Concrete in Compression.—Concrete does not possess the property of proportionality of stress and deformation for wide ranges of stress as does steel; in other words, the deformation produced by a load is not proportional to the compressive stress. The relation between stress and deformation is not entirely uniform; there are even considerable differences in deformation for the same mixtures.

Various curves have been proposed to represent the stress-deformation relation, but the parabola is the most satisfactory general representation. Frequently the parabola expresses the relation almost exactly, and in nearly every case the parabolic relation will fit the stress-deformation diagram very closely throughout the part which is ordinarily developed in beams, the lack of agreement near the crushing point not being of importance. The analytical work with the parabola is not complicated, and this curve offers easy comparison with the straight-line relation and easy translation to this relation. Even if the straight-line relation be accepted as sufficient for use with ordinary working stresses, the parabolic or other variable relation must be used in discussing experimental data when any considerable deformation is developed in the concrete. The parabola will be adopted as the basis of the analytical work used in this discussion.

Figure 186 shows such a stress-deformation curve. For purposes of illustration, the crushing strength of the concrete is represented as 2,000 lbs. per sq. in., and the ultimate unit deformations as .002. The relation between proportionate stress or ratio of stress developed to ultimate compressive strength of the concrete $\frac{c}{c'}$ and proportionate deformation or ratio of deformation developed at the given stress to ultimate or crushing deformation $\left(\frac{\lambda_c}{\lambda'_c} = q\right)$ which forms the basis of this analysis, is also shown by the figure.

Modulus of elasticity is a term which has been used very loosely in connection with reinforced concrete. In the general theory of flexure it is defined to be the ratio of the unit stress to the unit deformation within the elastic limit of the material.

As applied in this way to materials having the property of proportionality of stress and deformation, the modulus of elasticity is a constant. For materials with a variable stress-deformation relation like concrete it may not be considered proper to call the variable ratio the modulus of elasticity, and such a use in connection with formulas for flexure of concrete may lead to misunderstandings. However, it is important that a definite expression for this ratio be found. The writer obtains this relation from the initial modulus of elasticity, and uses the term "initial modulus of elasticity" to express the relation which would exist between stress and deformation if the concrete compressed uniformly at the rate it compresses for the lower stresses. The tangent of the angle which the line A C in Fig. 186 makes with the vertical gives this initial modulus of Elasticity E_c. The line is tangent to the parabola at A, and its equation is $x = E_c y$. By means of this initial modulus of elasticity the parabolic

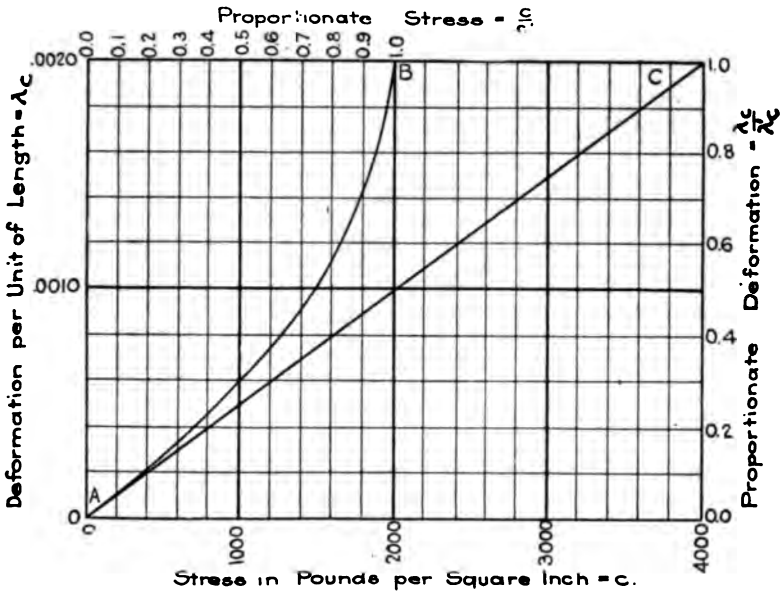


Fig. 186.—Stress Deformation Curves for Concrete in Compression.

stress-deformation relation may, from the properties of the parabola, be expressed as

$$c = E_c \lambda_c - \tfrac{1}{2} \frac{1}{\lambda'_c} E_c \lambda^2_c = (1 - \tfrac{1}{2} q) E_c \lambda_c \quad \ldots\ldots\ldots (1),$$

in which q is the ratio of the deformation developed to the ultimate or crushing deformation of the concrete. From this the following equation is also true:

$$\frac{c}{c'} = (1 - \tfrac{1}{2} q)\, 2\, q \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (2).$$

These relations are fundamental. The values of E_c, c, and λ_c

must be obtained experimentally. The line for E_c should be taken as the line which will give a relation which will best fit throughout the range used in the test of beams, and λ'_c should be taken as the abscissa of the vertex of the parabola which fits best, and not necessarily as the actual crushing deformation of the concrete. It is the general relation which is important and not the values at the point of failure. Many stress-deformation diagrams have been gone over in this way, and this representation has been found quite satisfactory. It may be noted from Fig. 186 that while 2,000 lbs. per sq. in. will give a deformation of .002, it will take 1,500 lbs. per sq. in. to produce one-half of that deformation. For small stresses the stress-deformation curve does not differ much from the line of initial modulus of elasticity.

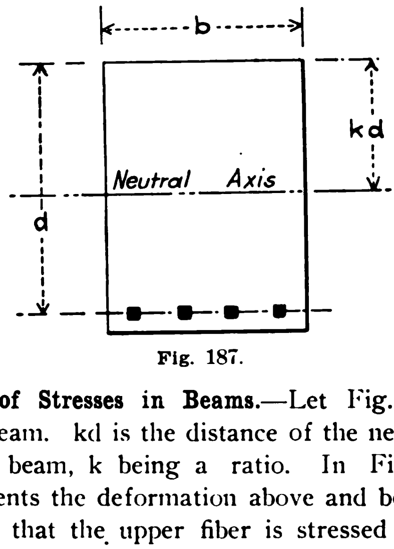


Fig. 187.

Distribution of Stresses in Beams.—Let Fig. 187 show the section of the beam. kd is the distance of the neutral axis below the top of the beam, k being a ratio. In Fig. 188, the left diagram represents the deformation above and below the neutral axis. Consider that the upper fiber is stressed to the point of failure; the upper deformation will then be the ultimate or crushing deformation. Since the deformations are proportional to the distances from the neutral axis, the curve of compressive stresses shown on the right will be a parabola with its vertex at O. The horizontal distances to the "line for initial modulus of elasticity" represent the stresses which would exist for the same deformation with a constant modulus of elasticity equal to E_c. The stress in the steel is represented by a length proportional to the ratio of the modulus of elasticity of the steel to the initial modulus of elasticity of the concrete $e = \frac{E_s}{E_c}$. In like

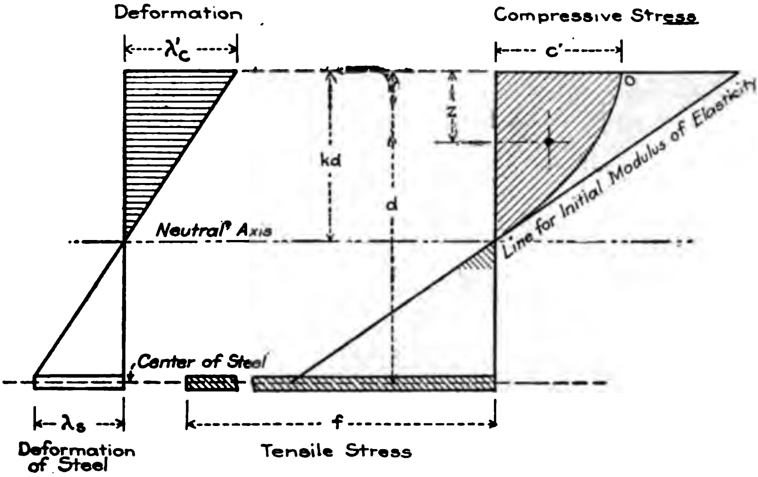


Fig. 188.—Stress and Deformation Distribution at Ultimate Deformation.

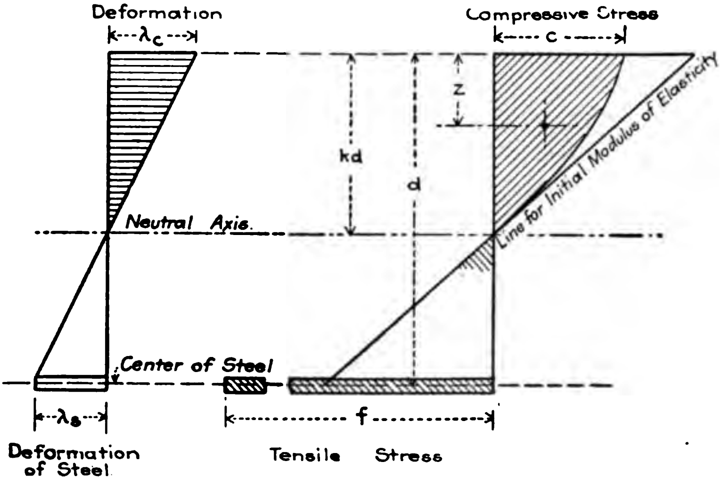


Fig. 189.—Stress and Deformation Distribution at Three-Fourths Ultimate Deformation.

manner Fig. 189 gives the stress and deformation distribution for a deformation of the upper fiber equal to three-fourths of the ultimate deformation of the concrete and a stress of fifteenxteenths of the crushing stress. Fig. 190 shows a similar distribution for one-half ultimate deformation and three-fourths crushing stress. It will be noted that the parabolic arc appears somewhat different from that in Fig. 188, and that it differs much less from the line for initial modulus of elasticity.

Relations in the Stress Diagram.—In deriving formulas for resisting moment, position of neutral axis, and compressive stress at upper fiber, three relations in the stress diagram are needed: (1) the relation of the stress c and the deformation λ_c at the upper fiber; (2) the total compressive stress, here called ΣX; and (3) the position of the center of gravity of the compressive

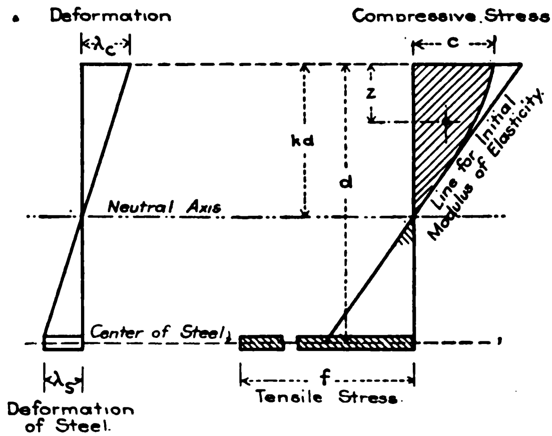


Fig. 190.—Stress and Deformation Distribution at One-Half Ultimate Deformation.

stresses given by the distance z. These relations vary for different values of the deformation in upper fiber. Basing the variation on the parabolic stress deformation law previously stated, and using $q = \frac{\lambda_c}{\lambda'_c}$ as the ratio of the deformation developed in the upper fiber to the ultimate deformation of the concrete, the following relations are readily deduced, though their derivation will not be given here.

$$\frac{c}{E_c\lambda_c} = 1 - \frac{1}{2} q \quad \text{.............................} (3).$$

$$\frac{\Sigma X}{\quad} = \frac{\text{Parabolic area}}{\text{Triangular area}} = 1 - \tfrac{1}{3} q \quad \text{......} (4).$$

$$\frac{z}{kd} = \frac{4 - q}{12 - 4q} = \tfrac{1}{3}\left(1 + \tfrac{1}{4}\,\frac{q}{3 - q}\right) \ldots\ldots\ldots\ldots (5).$$

Equation (3) gives the ratio of the compressive stress in the upper fiber to the stress which would exist for the same upper fiber deformation with a straight-line stress-deformation relation. Equation (4) gives the ratio of the summation of compressive stresses to the stress which would exist for the same upper deformation fiber with a straight-line stress-deformation relation. Equation (5) gives the ratio of the distance between the compression surface and the center of gravity of compressive stresses to the distance between that surface and the neutral axis.

Values for several ratios of the deformation developed in the upper fiber to the ultimate or crushing deformation of the concrete are given in the following table:

TABLE LXVII.

PROPERTIES OF THE STRESS DIAGRAM.

Property	At ultimate deformation	At ¾ ultimate deformation	At ½ ultimate deformation	At ¼ ultimate deformation	By straight line relation
	$q = 1$	$q = \frac{3}{4}$	$q = \frac{1}{2}$	$q = \frac{1}{4}$	$q = 0$
c	$\frac{1}{2} E_c\lambda_c$	$\frac{5}{8} E_c\lambda_c$	$\frac{3}{4} E_c\lambda_c$	$\frac{7}{8} E_c\lambda_c$	$E_c\lambda_c$
ΣX	$\frac{1}{3} E_c\lambda_c kbd$	$\frac{3}{8} E_c\lambda_c kbd$	$\frac{5}{12} E_c\lambda_c kbd$	$\frac{11}{24} E_c\lambda_c kbd$	$\frac{1}{2} E_c\lambda_c kbd$
z	$\frac{3}{8}$ kd	$\frac{13}{36}$ kd	$\frac{7}{20}$ kd	$\frac{15}{44}$ kd	$\frac{1}{3}$ kd
$\dfrac{c}{\dfrac{2pf}{k}}$	$\frac{3}{4}$	$\frac{5}{6}$	$\frac{9}{10}$	$\frac{21}{22}$	1

Figure 191 shows graphically the relations given by equations (3), (4) and (5). It will be seen that the center of gravity of the compressive stresses ranges from $\frac{3}{8}$ distance down to neutral axis (the value for a deformation equal to the ultimate deformation) to $\frac{1}{3}$ distance down to neutral axis at the lower limit, $\left(\text{ratio, } \dfrac{z}{kd}\right)$. The position for $q = \frac{3}{4}$ is $\frac{13}{36}$, equal to .36. This is not far from the value $\frac{4}{11}$ which has been used in the discussion of the experimental work, and which was obtained by another method of analysis. The position for $q = \frac{1}{4}$ is .341, and for $q = 0$ it becomes $\frac{1}{3}$ as in the straight-line relation. The other ratios are less nearly constant. The ratio for compressive stress at most remote fiber to that for direct proportionality with same deformation $\left(\dfrac{c}{E_c\lambda_c}\right)$ ranges from $\frac{1}{2}$ when ultimate deformation of concrete is developed to 1 for no deformation. The range for summation of compressive stress is from $^2/_3$ to 1. It should be remembered that these formulas are not applicable when tensile stresses of concrete need consideration.

Neutral Axis.—The foregoing relations and the analytical condition that the total horizontal compressive stresses and the total horizontal tensile stress are equal will, if tensile stresses in the concrete be neglected, readily enable the position of the

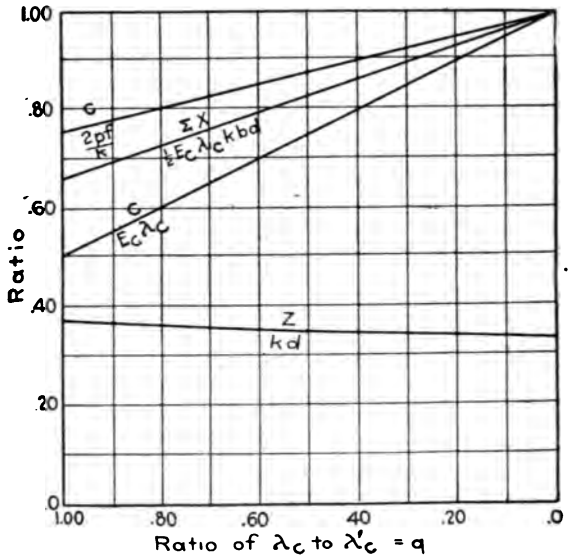


Fig. 191.—Diagram Showing Variation of Functions with q.

neutral axis to be determined for rectangular beams. From the proportionality of deformation (Figs. 188, 189 and 190).

$$\frac{\lambda_s}{1-k} = \frac{\lambda_c}{k} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6).$$

Equating horizontal stresses,

$$Af = \tfrac{1}{2}\left(1 - \tfrac{1}{3}q\right) E_c\lambda_c kbd \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (7).$$

Dividing (7) by (6) and substituting $f = E_s\lambda_s$,

$$AE_s(1-k) = \tfrac{1}{2}\left(1 - \tfrac{1}{3}q\right) E_c k^2 bd$$

Calling $\frac{E_s}{E_c} = e$ and $\frac{A}{bd} = p$,

$$pe(1-k) = \tfrac{1}{2}\left(1 - \tfrac{1}{3}q\right) k^2$$

Solving

$$k = \sqrt{\frac{2pe}{1 - \frac{1}{3}q} + \frac{p^2e^2}{\left(1 - \frac{1}{3}q\right)^2}} - \frac{pe}{1 - \frac{1}{3}q} \quad \ldots\ldots (8).$$

This gives the position of the neutral axis after tensile stresses in the concrete have become negligible and before the concrete reaches its ultimate strength. The value of k will vary slightly for the range of q usually considered, probably not more than .02.

For q = 1 equation (8) becomes:

$$k = \sqrt{3\,p\,e + {}^{9}/_{4}\,p^2e^2} - {}^{3}/_{2}\,pe \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9),$$

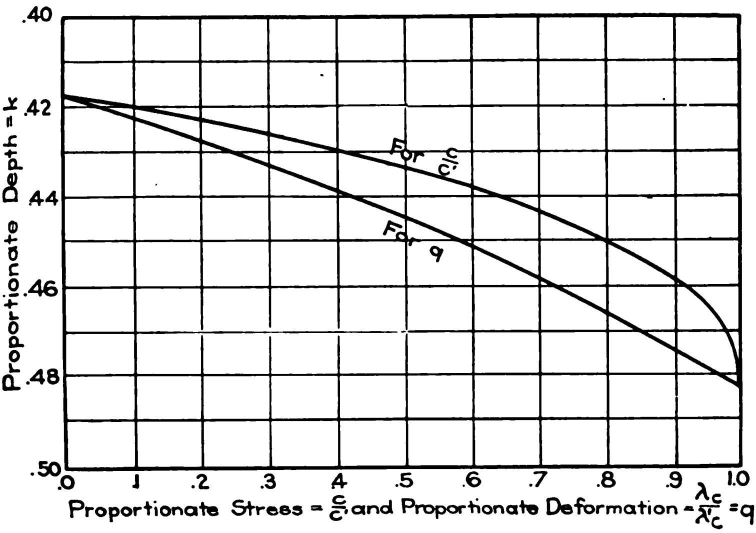


Fig. 192.—Variation in Position of Neutral Axis for Different Values of q

which is the expression when the concrete is at the limit of its compressive strength.

For q = 0, equation (8) becomes:

$$k = \sqrt{2\,pe + p^2e^2} - pe \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10),$$

which is the same as the value of k derived from a straight-line stress-deformation relation.

Fig. 192 shows the variation in k for e = 15 and a 1% reinforcement (p = .01), given both in terms of q and in terms of $\dfrac{c}{c'}$.

In this diagram the position of the neutral axis changes from .418 when q = 0 to .484 when the full or crushing deformation is developed. It shows a slow change for increasing values of the compressive stress until two-thirds of the full compressive strength of the concrete is developed. Beyond this the neutral axis lowers rapidly. Ordinarily a 1 per cent. reinforcement will not develop the full compressive strength of concrete, but the diagram serves to illustrate the change in the position of the neutral axis both in this and with other amounts of reinforcement. It is seen that the position remains nearly constant during what will be termed the third stage of beam action. Of course for low values of q, the tensile strength of the concrete would modify the position somewhat.

For the calculations in this paper and for the reinforcements used, k for q = ¼ gives results which are representative for the range considered, and will be used in the discussion. For q = ¼, equation (8) becomes

$$k = \sqrt{\frac{24}{11}pe + \frac{144}{121}p^2e^2} - \frac{12}{11}pe \quad \ldots\ldots\ldots\ldots\ldots\ldots(11).$$

This equation gives the position of the neutral axis for deformations which correspond closely with those developed under working stresses.

Figure 193 gives the position of the neutral axis based upon equation (11), (q = ¼) for e = 10, 12, 15 and 20. Calling the modulus of elasticity of steel, 30,000,000 lbs. per sq. in., these ratios correspond to initial moduli of elasticity of concrete of 3,000,000, 2,500,000, 2,000,000 and 1,500,000 lbs. per sq. in., respectively.

Resisting Moment.—When the tensile stresses in the concrete are neglected and the center of gravity of the compressive stresses is known, the value of the resisting moment of the beam (which it is readily seen is the moment of the couple formed by the tensile stress in the steel and the resultant of the compressive stresses in the concrete) is easily expressed as the product of the tensile stress in the steel and the distance from the center of the steel to the center of gravity of the compressive stresses. Hence the formula for the resisting moment for a rectangular beam is

$$M = Af\,(d - z) \quad \ldots\ldots\ldots\ldots\ldots\ldots(12).$$

It was shown that z varies slightly for different compressive stresses. The value of z when the concrete at the remote fiber is stressed three-fourths of its ultimate deformation ($q = \frac{3}{4}$) is approximately .36 kd; for $q = \frac{1}{2}$, .35 kd, and for $q = \frac{1}{4}$, .34 kd.

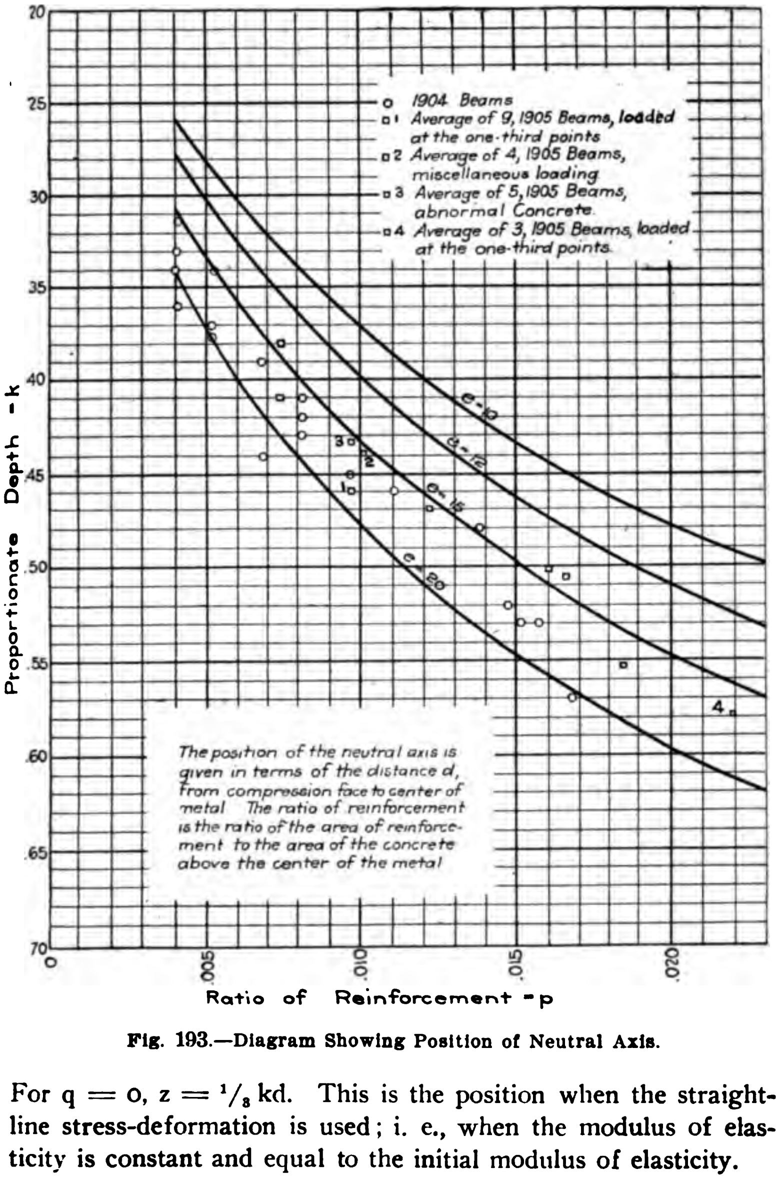


Fig. 193.—Diagram Showing Position of Neutral Axis.

For $q = 0$, $z = \frac{1}{3}$ kd. This is the position when the straight-line stress-deformation is used; i. e., when the modulus of elasticity is constant and equal to the initial modulus of elasticity.

When the E_c of the concrete is known and the amount of reinforcement is fixed, equation (12) will take the form

$$M = A\ f\ d' \ldots\ldots\ldots\ldots\ldots\ldots\ldots(13),$$

where d' is the moment arm of the couple and may be expressed as a proportionate part of d. Thus for $q = \frac{1}{4}$, with $E_c =$ 2,000,000 lbs. per sq. in. ($n = 15$) and 1% reinforcement ($p =$ .01), $d' = .853\ d$. For 1.5% reinforcement ($p = .015$), $d' =$ $.831\ d$. The values of the resisting moment for these reinforcements become $.853\ A\ f\ d$ and $.831\ A\ f\ d$, respectively.

This method offers the most convenient way of calculating the resisting moment so far as it is controlled by the tension of the steel within its elastic limit. The position of the neutral axis may well be taken from a diagram like Fig. 193, and the value of d' is then easily obtained.

Generally it will be best to use the resisting moment in terms of the tension in the steel, but if it is desired to express it in terms of the compression in the concrete the following equation may be used.

$$M = \left(\frac{1 - \frac{1}{3}q}{1 - \frac{1}{2}q}\right) \frac{1}{2}\ c\ k\ b\ d\ (d - z)\ \ldots..(14).$$

At least an approximate value of q will be known which may be used in equation (14). The fractional coefficient is the reciprocal of the function $\dfrac{c}{\frac{2pf}{k}}$ given in Fig. 191.

Compressive Stress at Upper Fiber.—The formulas for the position of the neutral axis and moment of resistance are based upon the assumption that the compressive stress in the upper fiber is within the crushing strength. To determine the value of the upper compressive stress substitute equation (3) in equation (7). This reduces to

$$c = \frac{2\ Af}{kbd} \cdot \frac{1 - \frac{1}{2}q}{1 - \frac{1}{3}q} = \frac{2\ pf}{k} \cdot \frac{1 - \frac{1}{2}q}{1 - \frac{1}{3}q} \ldots\ldots\ldots.(15).$$

For a deformation of upper fiber equal to three-fourths of the deformation at crushing $\left(\text{or } c = \frac{15}{16} c'\right)$, this becomes $c = \frac{5}{6} \frac{2pf}{k}$. For an upper deformation equal to one-half of ultimate deformation this becomes $c = \frac{9}{10}\,\frac{2pf}{k}$. For the crushing point of the concrete it becomes $c = \frac{3}{4}\,\frac{2pf}{k}$. As the upper deformation decreases, the value of c approaches $\frac{2pf}{k}$ which is the amount of the stress for a constant modulus of elasticity equal to the initial modulus of elasticity. By multiplying $\frac{2pf}{k}$, the stress found on the basis of a constant modulus of elasticity and a known position of the neutral axis, by this ratio $\frac{1 - \frac{1}{2}q}{1 - \frac{1}{3}q}$, the value of the compressive stress is found. The variation in this ratio may be seen in the upper line in Fig. 191 and also in the last line of Table LXVII. It will be seen that for high compressive stresses the stress developed is much less than that given by the straight-line relation using the value of the initial modulus of elasticity, being only three-fourths as much if the full compressive stress is developed. For low compressive stresses the discrepancy is much less.

It should be noted that when the compressive deformation developed is well up to the ultimate, the compressive stress calculated from equation (15) is much less than that found by using the formula $\frac{2pf}{k}$, (or any formula based on a straight-line stress-deformation relation), but when the load develops a deformation which is a small proportion of the ultimate, as may be the case for working loads, the coefficient found in equation (15) will not differ much from unity, and the straight-line formula will be but little in error.

When it is desired to check or design a beam, using the above theory, the following relations should be kept in mind. As explained above, in connection with equation (13), q may be taken equal to $\frac{1}{4}$ the ultimate deformation, which condition will exist within the limit of the usual working stresses in the concrete. Then from Table LXVII., z will equal $^{15}/_{44}\,kd = .34\,kd$, the value of k being taken from Fig. 193 for the desired ratio of reinforcement p and ratio between moduli of elasticity of the steel and concrete $\frac{E_s}{E_c} = e$.

Then d, the effective depth of the beam, being assumed, the value of $z = .34\,kd$ becomes known, and $d - z = d'$, which is the lever arm between the centers of gravity of the compression and tension forces. Equation (13) may then be solved.

Example.—Design a beam of sufficient section to develop a resisting moment of 234,375 in. lbs., using 1 per cent. of reinforcement and a unit stress of 16,000 lbs. per sq. in. on the metal.

Our equation (13) is

$$M = A\,f\,d';$$

then

$$A = \frac{M}{fd'} = \frac{234{,}375}{16{,}000\;d'}.$$

$$d' = d - z. \quad \text{Assume } d' = 12 \text{ ins.}$$

Now $z = .34\;k\,d$.

From Fig. 192, assuming $e = 10$,

$$k = .37.$$

Then

$$z = .34 \times .37\,d = .126\,d.$$

$$d' = d - .126\,d = .874\,d.$$

But

$$d' = 12 \text{ ins.},$$

and

$$d = 14.2 \text{ ins.} = \text{effective depth of beam.}$$

Then

$$A = \frac{234{,}375}{16{,}000 \times 12} = 1.22 \text{ sq. in.}$$

$$p = .01 = 1.22 \text{ sq. in.}$$

Then

$$A_c = \text{area of concrete} = 122 \text{ sq. ins.};$$

but

$$d = 14.2.$$

Then b, the width of the beam, $= \frac{122}{14.2} = 8.6$ ins.

We will therefore have a beam 8.6 ins. wide, 14.2 ins. + 1.8 = 16 ins. deep, assuming that the bars have 1.8 inches of concrete below their centers; two ⅞-in. diam. bars = 1.20 sq. ins., which will be used for the reinforcement.

Wason's Formula.—Mr. Leonard C. Wason, Assoc. M. Am. Soc. C. E., presented an extremely simple formula for the solution of reinforced concrete beams, in the Transactions of the American Society of Civil Engineers, Vol. 46, page 102. This formula is among the earliest used in this country and has the merit of having been long and extensively used. It is stated that beams designed by it when tested had a factor of safety varying from 3 to 5.

The formula is based on the following assumption: That there is a perfect bond between the steel and the concrete within the limits of the working stresses of the combination. That the

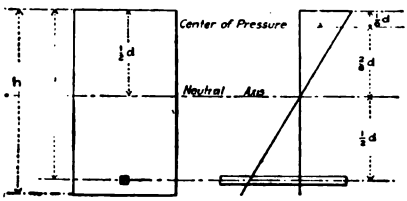


Fig. 194.—Diagram Illustrating Beam Formula of Mr. L. C. Wason.

steel takes the entire tensile stress and the concrete the entire compressive stress. That the neutral axis is assumed to be half way between the center of the reinforcing bars and the top of the beam. That the center of pressure of the concrete under compression is considered as being two-thirds of the height from the neutral axis to the top of the beam. The distance from the center of pressure of the concrete in compression to the center of the reinforcement equals $^5/_6$ d. In Fig. 194 let

d = effective depth of beam.

l = span in inches.

F_s = total stress in steel.

W = total uniform load in pounds.

Then, taking the center of pressure as the center of moments, the resisting moment

$$M = \tfrac{5}{6}\, d\, F_s.$$

The bending moment of a beam for a uniformly distributed load

$M = \frac{1}{8} W l$. Equating these two moments and solving for f_s we obtain

$$F_s = \frac{Wl}{6\frac{2}{3} d}.$$

Mr. Wason states that the above assumption for the position of the neutral axis is somewhat higher than its position according to theoretical assumption, but that it approximates closely the position as found by actual determinations during the test of beams.

Example.—Determine amount of steel required for a beam of 12.5 ft. span to carry a total uniform load of 12,500 lbs., assuming effective depth determined in example on page 308, of 14.4 ins., and using a unit stress for the steel of 16,000 lbs. per sq. in.:

$$12.5 \text{ ft.} = 150 \text{ inches.}$$

$$F_s = \frac{12{,}500 \times 150}{6\frac{2}{3} \times 14.4} = 19{,}500 \text{ lbs.}$$

$$\frac{19{,}500}{16{,}000} = 1.22 \text{ sq. ins.}$$

2 bars $\frac{13}{16} \times \frac{13}{16}$-in. give an area of 1.32 sq. ins.

After determining the total stress in the metal, the area of the reinforcement is determined by dividing the total stress by a safe working stress to determine the area of metal. Bars of proper size are selected to make up this area, a convenient spacing selected, and the area of the concrete adjusted to resist the compression. Mr. Wason uses 16,000 lbs. per sq. in. tension on the steel, and for a 1:3:6 concrete an average of 500 lbs. per sq. in. in compression on the concrete, and requires 32 sq. in. of concrete in the upper third of the beam for each square inch of steel. This averages very nearly 1 per cent. of reinforcement. The above ratios are applied to the use of Ransome twisted bars, which have, as explained on page 227, a high elastic limit and give a factor of safety of about 4. In the above problem the total compression is 19,500 lbs.; this, divided by 500 lbs., gives a required area in the upper third of the beam of 39.00 sq. ins., $39.0 \times 3 = 117$ sq. ins., total area of beam, 117 sq. ins. $\div$ 14.4 ins. depth assumed gives 8.13 ins. width of beam; a width of $8\frac{1}{4}$ ins. may be used.

Ransome's Formula.—Ransome's formula for a simple beam uniformly loaded is

$$S = \frac{Wl}{7d}$$

in which

W = total dead and live loads in tons.
l = span in inches.
d = depth of steel below top of beam = effective depth.
S = maximum stress in beam, either tension or compression.

When the beam is not uniformly loaded the formula becomes

$$S = \frac{B\,M \times 8}{7\,d}$$

in which B M equal the maximum bending moment in inch tons.

In order that the compressive stress per lineal foot resulting from a chosen value for d shall not exceed the safe compressive strength of the concrete there must be 16 sq. ins. of concrete above the bars for each ton of stress.

$$16\,S = 12\,d,$$

from which

$$S = \tfrac{3}{4}\,d.$$

Substituting this value of S in the above formula we have

$$\tfrac{3}{4}\,d = \frac{W\,l}{7\,d}$$

$$d = \sqrt{\frac{4}{21}\,W\,l.}$$

Having obtained d, the total stress in tons $S = \tfrac{3}{4}\,d$.

Example.—Assume a flat floor slab, having a span of 12 ft. carrying a live load of 150 lbs. per sq. ft.

It is necessary to assume the dead weight of the floor. Let this be taken at 75 lbs. per sq. ft., making a total load of 225 lbs. per sq. ft. The total load W in tons on a strip of floor 1 ft. wide would be

$$\frac{12 \times 225}{2{,}000} = 1.35 \text{ tons},$$

and we have for d,

$$d = \sqrt{\frac{4 \times 1.35 \times 12 \times 12}{21}} = 6.08 \text{ inches.}$$

The total stress in the bars would equal $\tfrac{3}{4} \times 6.08 = 4.56$ tons. Assuming an allowable working stress on the metal of 8 tons per sq. in., there will be .57 sq. ins. of metal required, or 4 rods 3/8 in. square in each foot width of slab. When 1/4-in. rods are used the distance from center of rod to bottom should be at least 1/2 in., and 3/4 in. for 1/2 in. square rods. For 3/8-in. rod reinforcements we will have a total thickness of 6 3/4 ins.

Ribbed Floors.—In calculating the beam dimension and amount of reinforcement for ribbed slabs, the formula

$$S = \frac{W\,l}{7\,d}$$

is used. This condition, however, is imposed, that the upper third of the beam, including the flat slab connecting the ribs, shall contain at least 5 sq. ins. of concrete for each ton of stress given by the formula. This condition prevents the concrete in the top of the slab from being strained beyond its safe compressive strength.

Example.—Assume a ribbed floor of 20 ft. span, loaded with 200 lbs. per sq. ft., to find dimensions of floor and size of reinforcing bar. We will assume the weight of the floor to be 60 lbs. per sq. ft., making the total load 260 lbs. per sq. ft. It is necessary to fix on some spacing of the beams; this is usually taken from 3 to 4 ft. centers.

We will take 4 ft. between centers for our beam spacing, and the total load on each beam will be

$$L = \frac{20 \times 4 \times 260}{2{,}000} = 10.4 \text{ tons.}$$

and the total stress is

$$S = \frac{10.4 \times 20 \times 12}{7 \times d} = \frac{356.2}{d}.$$

If we use a 1 1/4 in. × 1 1/4 in. bar having an area of 1.5625 sq. ins., and a safe tensile strength of 10 tons per sq. in., the depth d will be

$$d = \frac{356.2}{1.5625 \times 10} = 22.7 \text{ ins.}$$

The minimum thickness of slab to give the required compressive strength will be

$$\frac{5 \times 1.5625 \times 10}{48} = 1.63 \text{ ins.}$$

This will be less than it will be advisable to use, as from a practical standpoint a thickness of from 2 to 3 ins., at the least, should be used. The slab should be figured as a flat floor spanning from beam to beam. Thus, if the slab be unreinforced, and we assume a unit tensile working strength of 50 lbs. per sq. in. in the concrete, and if the rib be assumed at 4 ins. thick, we have a clear span of 44 ins. The external moment in in.-lbs. will then be

$$M = \frac{260 \times 44 \times 44}{12 \times 8} = 5{,}244 \text{ in. lbs.}$$

For a rectangular beam the resisting moment

$$M = S \frac{I}{c}$$

$$S = 50$$

$$\frac{I}{c} = {}^1/_6\, bd^2 = \frac{12 \times d^2}{6} = 2\, d^2$$

and

$$M = 100\, d^2 = 5{,}244 \text{ in. lbs.,}$$

and

$$d = 7.25 \text{ ins.,}$$

which will be the proper thickness of an unreinforced slab between beams spaced 4 ft. center for the live loading of 200 lbs. per sq. ft. The assumed dead load was, therefore, too low. For such a heavy live load a reinforced slab should be used.

The width of beam b and total height h depend upon the size of the bar used and are given in the following table as used by Ransome. Dimensions are given in inches.

Size of bar	½ in.	¾ in.	1 in.	1¼ in.	1½ in.	1¾ in.	2 in
h	d + ¾	d + 1⅛	d + 1½	d + 1⅞	d + 2¼	d + 2⅝	d + 3
b	1⅛	2¼	3	3¾	4⅛	5¼	6

Where ribbed floors of long spans are used it is often advisable to run one or more stiffening ribs at right angles to the main beams. These will stiffen the longitudinal beams laterally and assist in the distribution of stresses in case any heavy concentrations are brought upon the floor system. These stiffening beams should be reinforced at the bottom, and it is customary to rest the bar directly upon the main bar where they cross.

Cantilever Beams.—If a beam is supported at one end only, the stress in the beam is four times that in a beam of equal span supported at both ends. Hence, the formula for cantilever beams uniformly loaded is

$$\text{Stress} = \frac{4\ l'W}{7\ d}$$

when l' = the length or projection of the beam.

In this case the tension bar is placed in the top of the beam and the portion below the bar must contain at least 16 sq. ins. of concrete for each ton of stress.

Wall and Pier Footings.—The general form and arrangement of the Ransome wall and column footing is shown in Figs. 195 and 196. In all cases the width of wall T and load per lineal

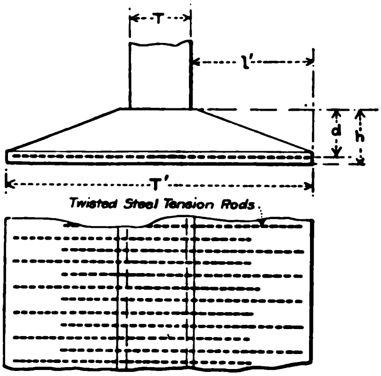


Fig. 195.—Typical Wall Footing by Mr. E. L. Ransome.

foot W and the width of footing T′ will be given. The total stress in the tension bars or the total compression in the concrete per lineal foot is

$$S = \frac{2\ W\ l'}{7\ d}$$

in which W equals the total load in tons, l' equals the projection in inches, and d equals the distance in inches from the top of the footing to the center of the bars. There are two unknown quantities, stress S and d. It is therefore necessary to impose another condition, and it is that when the safe compressive strength of the concrete equals 35 tons per sq. ft. there shall be 16 sq. ins. of concrete in the area above the bars for each ton of stress, or $16 \times S = 12\ d$, from which $S = \frac{3}{4}\ d$. This condition is necessary in order that the concrete shall not be strained beyond its safe compressive strength, and should be modified to suit the strength when the latter does not conform to the value of 35 tons. Substituting this value of S in the above formula and reducing, we have

$$d = \sqrt{\frac{8\,W\,l'}{21}}$$

Having obtained d from this formula, the total stress S in the bars in tons = ¾ d. The bars should be arranged as shown in Fig. 195. The sizes of the bars should be so taken that the bars

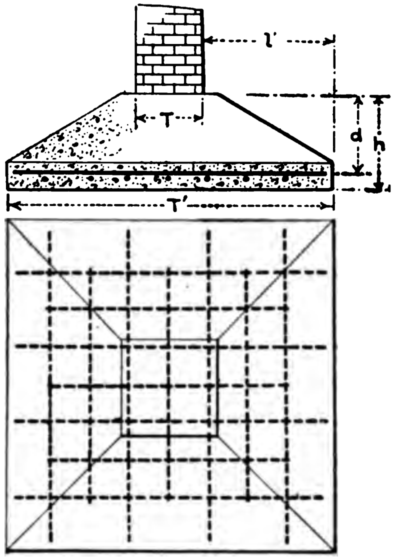


Fig. 196.—Typical Column Footing by Mr. E. L. Ransome.

will not be spaced more than 12 ins. apart. The total height h of the footing should be at least 3 ins. greater than the depth d.

Example: Let T = 2 ft., W = 30 tons, and the safe bearing power of soil equal 2.5 tons. Then T' = 12 ft. and l' = 5 ft. = 60 ins., and

$$d = \sqrt{\frac{8 \times 30 \times 60''}{21}} = 26.2 \text{ ins.} = 26\tfrac{1}{4} \text{ ins., approximately;}$$

and stress S = ¾ × 26.2 = 19.65 tons, requiring ¾-inch square bars, $3\frac{7}{16}$ ins. centers, assuming a working stress of 10 tons. per sq. inch for the steel. Then the length will be

$$T' - \frac{l'}{2} = 9\tfrac{1}{2} \text{ ft.}$$

Formulæ for Pier Footings.—As in the case of wall footings we have given, the dimensions of the piers to be supported and of the footing T and T′ and the total load W to be carried.

The formula for obtaining the total stress S in the tension bars running in each direction is

$$S = \frac{W\,l'}{3\,d}$$

in which we have as before the two unknown quantities, S and d. In order that the concrete may not be stressed in compression beyond its safe working strength, we impose the condition

$$4\,S = \frac{d}{2}\,(T + 6),$$

from which

$$S = \frac{d\,(T+6)}{8}.$$

Substituting this value of S in the above formula and reducing we have

$$d = \sqrt{\frac{8\,W\,l'}{3\,(T + 6)}}$$

Having obtained d by this formula, the total stress S in tons will be

$$S = \frac{T + 6}{8} \times d,$$

from which the size and number of bars running in each direction can be computed. These bars may be in two lengths, as shown in Fig. 196, the shorter length being equal to T + l′. The total height H should not be less than d + 4 ins.

Example.—Let T = 20 ins., load W = 100 tons. Safe bearing power of soil equals 2 tons per sq. ft. The required area of base of footing is 50 sq. ft. and the width T′ equals the square root of 50 = 7 ft. 0⅞ ins., or 85 ins. approximately, and l′ = 32.5 ins.

$$d = \sqrt{\frac{8 \times 100 \times 32.5}{3 \times 26}} = 18.5 \text{ inches.}$$

The total stress will be

$$S = \frac{18.5 \times 26}{8} = 60 \text{ tons.}$$

requiring 11, ¾-in. bars, or 24, ½-in. bars running in each direction. These bars should be spaced equally over area T′, giving for ¾-in. bars a spacing of about 7 ins. and for ½-in. bars of about 3½ ins. The total height h = 18.5 + 4 = 22½ ins.

Thacher's Formulas.—Thacher's empirical formulas are based on the ultimate safe loads, and have been found to agree quite closely with the results obtained by experiment. These formulas are quite easy to apply and give safe and satisfactory results. The formulas developed below are for ultimate strength, and a factor of safety depending upon the conditions and judgment of the designer should be used.

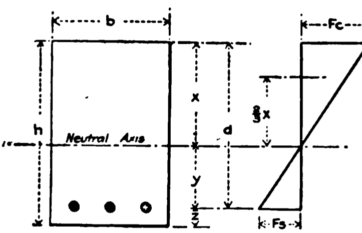


Fig. 197.—Thacher's Formula, Rectangular Beams, Single Reinforcement.

Let F_c = probable crushing strength of the concrete per sq. in. in pounds.
f_c = compressive stress in pounds per sq. in. on concrete.
F_s = tensile strength per sq. in. on steel = ultimate strength per sq. in. on test piece + 10 per cent.
f_s = tensile stress in pounds per sq. in. on steel.
E_c = modulus of elasticity of concrete.
E_s = modulus of elasticity of steel.
$e = \dfrac{E_s}{E_c} = 20$, as taken by Mr. Thacher.
A = area of steel in tension for 1 inch in width of beam.
l = length of beam in feet = span.
M = bending moment in foot pounds for 1 inch in width of beam (ultimate).
W = load at center, including weight of beam for 1 inch in width (ultimate).
w = uniform load per lineal foot, including weight of beam for 1 inch width (ultimate).
w′ = 12 w = uniform load per sq. ft. (ultimate).
b, h, d, x, y and z are as shown in Fig. 197.

First Case.—Assume a rectangular beam, with reinforcement in tension side only (Fig. 197), and neglecting the tensile strength of the concrete.

Assuming that plane sections before bending are plane after bending, we have the relation

$$x : \frac{F_c}{E_c} : y : \frac{F_s}{E_s}.$$

Therefore

$$x = y\,e\,\frac{F_c}{F_s} \quad \text{......................} \quad (T\,1)$$

or

$$F_c = \frac{F_s\,x}{e\,y} \quad \text{......... ....} \quad (T\,2)$$

and

$$d = x + y = y\,e\,\frac{F_c}{F_s} + y,$$

from which

$$y = \frac{d}{e\,\dfrac{F_c}{F_s} + 1} \quad \text{.....................} \quad (T\,3)$$

also

$$x = d - y \quad \text{.......................} \quad (T\,4)$$

In order that there be equilibrium, the total compression must equal the total tension, and we have

$$\frac{F_c\,x}{2} = A\,F_s \text{ or } x = 2\,\frac{A\,F_s}{F_c} \quad \text{..............} \quad (T\,5)$$

Substituting the value of F_c found in equation (T 2) in equation (T 5), we have

$$x^2 = 2\,A\,e\,y.$$

and

$$y = \frac{x^2}{2\,A\,e}$$

Substituting the value of x from equation (T 5), we have

$$y = \left[\left(\frac{F_s}{F_c}\right)^2\left(\frac{2}{e}\right)\right] A$$

$$d = x + y = 2\left[\frac{F_s}{F_c} + \frac{1}{e}\left(\frac{F_s}{F_c}\right)^2\right] A$$

$$\text{therefore } A = \frac{d}{2\left[\dfrac{F_s}{F_c} + \dfrac{1}{e}\left(\dfrac{F_s}{F_c}\right)^2\right]} \quad \text{..............} \quad (T\,6).$$

Now

$$d = x + y = x + \frac{x^2}{2\,A\,e},$$

and

$$x^2 + 2\,A\,e\,x = 2\,A\,e\,d$$

from which

$$x = \sqrt{2\,A\,e\,d + (A\,e)^2} - e\,A \quad \text{................} \quad (T\,7)$$

Let M = resisting moment of beam. Then taking

M_c = resisting moment of concrete in compression.

M_s = resisting moment of steel in tension.

Taking the neutral axis of the beam as the center of moments, x and y are in inches, and

$$M_c = \frac{F_c x}{2} \times \tfrac{2}{3} x = \frac{F_c x^2}{3},$$

Dividing by 12 to reduce to foot-pounds,

$$M_c = \frac{F_c x^2}{36}.$$

and $$M_s = F_s Ay \text{ in inch-pounds,}$$

or, $$M_s = \frac{F_s Ay}{12} \text{ in foot-pounds.}$$

The total resisting moment of the beam will be $M = M_c + M_s$.

$$M = \frac{F_c x^2}{36} + \frac{F_s Ay}{12}.$$

Substituting for F_c its value from equation (T 2), we have

$$M = \frac{F_s x^3}{36\, e y} + \frac{F_s Ay}{12}$$

$$= \frac{F_s}{36}\left(\frac{x^3}{e y} + 3\ A y\right) \qquad \text{(T 8)}$$

For a simple beam with uniform load

$$M = \tfrac{1}{8}\, w\, l^2 \text{ and } w = \frac{8M}{l^2}.$$

Therefore

$$w = \frac{F_s}{4.5\ l^2}\left(\frac{x^3}{e y} + 3\ A y\right) \qquad \text{(T 9)}$$

and

$$w' = 12\ w = \frac{12\ F_s}{4.5\ l^2}\left(\frac{x^3}{e y} + 3\ A y\right)$$

For a simple beam loaded at the center,

$$M = \tfrac{1}{4}\ W l,$$

and

$$W = \frac{F_s}{9\ l}\left(\frac{x^3}{e y} + 3\ A y\right) \qquad \text{(T 10)}$$

To check a simple beam, assume proper values for F_s, E_s, E_c; A, d and l are known. Then equation T7 gives the value of x, and equations (T 8), (T 9) and (T 10) will give M, W and w, as may be required. Then if the value of F_c from equation (T 2) exceeds the assumed or determined ultimate strength of the concrete in compression the values of M W or w, as determined by equations (T 8), (T 9) and (T 10); should be reduced in the ratio that the probable ultimate compressive strength of the concrete bears to F_c, as found by equation (T 2).

To design a beam, values must be assigned for F_s, F_c, E_s, E_c, h, z and d; l will usually be found by conditions of the problem, as is also sometimes h; z should usually be from $1\frac{1}{2}$ to 2 times the diameter of the rods. If h or d is not fixed by existing conditions, a value should be selected by trial to give an economic

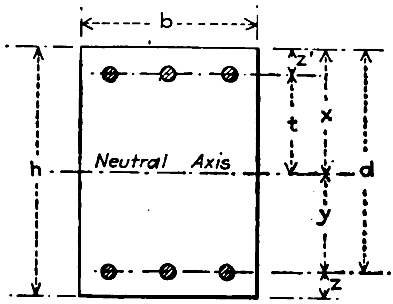


Fig. 198.—Thacher's Formula, Rectangular Beams, Double Reinforcement.

design. Mr. Thacher usually assumes e equal to 20. The equation (T 3) gives

$$y = \frac{d}{e\dfrac{F_c}{F_s} + 1} \quad \text{and } x = d - y.$$

From equation (T 6),

$$A = \frac{d}{2\left[\dfrac{F_s}{F_c} + \left(\dfrac{F_s}{F_c}\right)^2 \times \dfrac{1}{e}\right]}$$

and the spacing of rods c. to c. will equal $\dfrac{\text{area of rod}}{A}$.

M and W or w may be found directly from equations (T 8), (T 9) and (T 10).

Second Case.—Assume a rectangular beam reinforced in both tension and compression sides (Fig. 198), and neglecting the tensile strength of the concrete.

Let F_s^c = compression stress in pounds per sq. in. in steel.

A_s^c = area of steel in compression for 1 in. width of beam.

$t = x - z'$.

As before:

$$x : \frac{F_c}{E_c} : : y : \frac{F_s}{E_s} : : t : \frac{F_s^c}{E_s},$$

from which

$$x = \frac{F_c}{F_s} e y \text{ and } F_c = \frac{F_s x}{e y},$$

and

$$F_s^c = \frac{t F_c e}{x}, \text{ also } F_s^c = \frac{F_s t}{y}.$$

Substituting the value of F_c above, in the last equation we have

$$F_s^c = \frac{F_s t}{y} \text{ and } y = \frac{F_s t}{F_c} \quad \text{.................. (T 11)}$$

$$d = x + y = \frac{F_c}{F_s} e y + y,$$

or

$$y = \frac{d}{\frac{F_c}{F_s} e + 1} \text{ and } x = d - y.$$

As the total tension must equal the total compression,

$$\frac{F_c x}{2} + A_s^c F_s^c = A F_s,$$

or

$$x = \frac{2 A F_s - 2 A_s^c F_s^c}{F_c} \quad \text{................ (T 12)}$$

Let $n = \frac{A_s^c}{A}$, and $A_s^c = n A$, and substituting the value of F_s^c from eq. (T 11) we have

$$A = \frac{F_c x y}{2 F_s (y - n t)} \quad \text{.................. (T 13)}$$

Substituting the value of F_c given above, and writing for d its value $x + y$,

$$d = \frac{x^2}{2 A e} + n t + x'.$$

Therefore

$$x = \sqrt{2 Ae (d + nz') + [Ae (n + 1)]^2} - [Ae (n + 1)]. \text{ . (T 14).}$$

For concrete:

$$M = \frac{F_c x^2}{36}.$$

For the steel in compression:

$$M = \frac{F_s^c A_s^c t}{12} = \frac{F_s^c n A_s t}{12}.$$

For the steel in tension:

$$M = \frac{F_s A_s y}{12},$$

and the total resisting moment

$$M = \frac{F_c x^2}{36} + \frac{F_s^c n A_s t}{12} + \frac{F_s A_s y}{12}.$$

Substituting the value of F_s^c from eq. (T 11) and of $F_c = \frac{F_s x}{e y}$, we have

$$M = \frac{F_s}{36}\left(\frac{x^3}{e y} + \frac{3 n A t^2}{y} + 3 A y\right) \ldots\ldots\ldots\ldots (T\ 15)$$

For a simple beam uniformly loaded $M = \frac{w l^2}{8}$ and $w = \frac{8 M}{l^2}$; therefore

$$w = \frac{F_s}{4.5 l^2} \cdot \left(\frac{x^3}{e y} + \frac{3 n A t^2}{y} + 3 A y\right) \ldots\ldots\ldots\ldots\ldots (T\ 16)$$

$$w' = 12 w = \frac{12 F_s}{4.5 l^2}\left(\frac{x^3}{e y} + \frac{3 n A t^2}{y} + 3 A y\right) \ldots\ldots\ldots (T\ 17)$$

For a simple beam loaded at the center, W being the total load,

$$W = \frac{4 M}{l},$$

therefore

$$W = \frac{F_s}{9 l}\left(\frac{x^3}{e y} + \frac{3 n A t^2}{y} + 3 A y\right) \ldots\ldots\ldots (T\ 18)$$

To check a beam, values must be found or assumed for F_s, e, h, z, z_1, d, and then from eq. (T 14),

$$x = \sqrt{2 Ae (d + nz') + [Ae (n + 1)]^2} - [Ae (n + 1)].$$

A_s = total area of tensile reinforcement divided by the width of the beam in inches, and $A_s^c = n A_s$, Equation (T 11) gives

$$F_s^c = \frac{F_s t}{y},$$

and Equation (T 2) gives

$$F_c = \frac{F_s x}{e y}.$$

If this value of F_c exceeds the probable ultimate resistance of the concrete to compression, the values of M and W or w found by equations (T 15), (T 16), (T 17) and (T 18) should be reduced in the ratio that the assumed compressive strength of the concrete bears to the value found by equation (T 2).

To Design a Beam.—Determine or assume values for F_s, F_c, e, h, d, z, z^1 and l as before. Then equation (T 3) gives

$$y = \left(\frac{d}{e \frac{F_c}{F_s}} + 1 \right)$$

and
$$x = d - y.$$

Equation (T 13) gives $A = \dfrac{F_c\, x\, y}{2\, F_s\, (y - n\, t)}$.

Equation (T 11) gives $F_s^c = \dfrac{F_s\, t}{y}$,

and Equations (T 15), (T 16), (T 17) and (T 18) give values of M, w, w' and W as required.

Third Case.—Mr. Thacher's analysis for the solution of a reinforced T section for a ribbed floor construction is as follows:

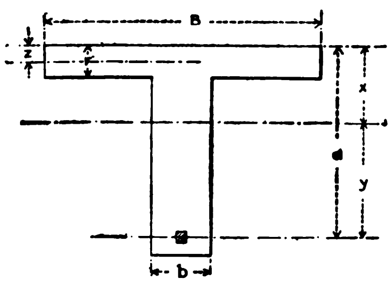


Fig. 199.—Thacher's Formula for T-Beams.

The slab is first designed as a beam with a span equal to the distance between floor ribs and with the reinforcing bars at right angles to the direction of the length of the joists. The joist spacing varies with the conditions, and when the spacing is not fixed by limiting conditions the spacing of the ribs, thickness of floor slab and depth of ribs should be so chosen as to give the most economic floor. Several solutions may be necessary to secure this result. In Fig. 199 let B = width of slab = distance center to center of ribs in inches, r = thickness of slab in inches. $A_s^{c'}$ represents the area of steel which is necessary to develop the same compressive resistance as the wings of the T. This is an imaginary quantity, as no steel reinforcement is used for reinforcing the slab in a longitudinal direction. Mr.

takes $z = \frac{r}{2}$ when r is the thickness of the slab.

To Design a Simple Beam.—Having assumed or d(
values for B, r, z, and assumed values for F_s, F_c, e, h
in the previous cases, and noting that

$$\frac{A_s^{c\prime}}{A_s} = n, \text{ and } \frac{A_s}{b} = A,$$

Then

$$A_s^{c\prime} = \frac{(B - b)\, r}{e};$$

$$y = \frac{d}{e \frac{F_c}{F_s} + 1}; \qquad x = d - y;$$

$$F_s^c = \frac{F_s t}{y};$$

$$A_s = \frac{F_c b x + 2\, A_s^{c\prime} F_s^c}{2\, F_s};$$

$$M = {}^1/_{12}\, [1/3\, F_c b x^2 + F_s^c A_s^{c\prime} t + F_s A_s y];$$

$$W = \frac{1}{3 l}\, (1/3\, F_c b x^2 + F_s^c A_s^{c\prime} t + F_s A_s y);$$

$$w' = \frac{8}{B l^2}\, (1/3\, F_c b x^2 + F_s^c A_s^{c\prime} t + F_s A_s y)$$

If M, W or w′ is greater than required, A_s can be
tionately.

To check a ribbed beam supported at the ends as
Assume values of F_s and e as before. Then

$$A_s^{c\prime} = \frac{(B - b)\, r}{e};$$

A_s = area of reinforcing bars;

$$x = \sqrt{2 A e (d + n z) + [A e (n + 1)]^2}$$

$$y = d - x; \qquad F_s^{c\prime} = \frac{t F_s}{y};$$

$$F_c = \frac{F_s x}{e y}.$$

Thacher's Constants.—The formula for the design of beams can be much simplified by substituting for F_c, F_s, E_c and E_s their values and reducing. The following table gives the values of A_s, M, W, w, w′ and d at 1 and 6 months for the following values:

E_s =	30,000,000
F_s, ultimate	64,000
E_c, for 1:2:4 concrete, one month old	1,460,000
F_c, for 1:2:4 concrete, one month old	2,400
E_c, for 1:2:4 concrete, six months old	2,580,000
F_c, for 1:2:4 concrete, six months old	3,700
E_c, for 1:3:6 concrete, one month old	1,220,000
F_c, for 1:3:6 concrete, one month old	2,050
E_c, for 1:3:6 concrete, six months old	1,860,000
F_c, for 1:3:6 concrete, six months old	3,100

THACHER'S CONSTANTS FOR BEAMS.

Proportions of Concrete	1:2:4		1:3:6	
Age of Concrete	1 month	6 months	1 month	6 months
Area of steel sq. ins. required for 1 in. width of beam = A	$\frac{d}{142}$	$\frac{d}{100}$	$\frac{d}{165}$	$\frac{d}{109}$
Ultimate bending moment ft.-lbs. for 1 in. width of beam = M	$35.62\ d^2$	$51.25\ d^2$	$30.62\ d^2$	$46.25\ d^2$
Breaking weight at center for 1 in. width of beam = W	$\frac{142.5\ d^2}{l}$	$\frac{205.0\ d^2}{l}$	$\frac{122.5\ d^2}{l}$	$\frac{185.0\ d}{l}$
Breaking weight per lineal ft. uniformly distributed for 1 in. width of beam = w	$\frac{285.0\ d^2}{l^2}$	$\frac{410.0\ d^2}{l^2}$	$\frac{245.0\ d^2}{l^2}$	$\frac{370.0\ d^2}{l^2}$
Breaking weight per sq. ft. uniformly distributed = w′	$\frac{3420\ d^2}{l^2}$	$\frac{4920\ d^2}{l^2}$	$\frac{2940\ d^2}{l^2}$	$\frac{4440\ d^2}{l^2}$
Effective depth of beam required for a uniform load of w′ lbs. per sq. ft. = d	$\sqrt{\frac{l^2\ w'}{3420}}$	$\sqrt{\frac{l^2\ w'}{4920}}$	$\sqrt{\frac{l^2\ w'}{2940}}$	$\sqrt{\frac{l^2\ w'}{4440}}$

The above formulas are for ultimate strength and it is necessary to divide the constants by the proper factor of safety to secure the desired working values. If calculations are based on formulas for concrete 1 month old there will not be sufficient reinforcing metal to develop the full strength of the concrete after the latter gains its full strength. It is therefore preferable to use formulas for concrete 6 months old, using such a factor of safety in 6 months as will give any desired factor of safety in 1 month. A factor of safety of 5 in 6 months will give a factor of about 3.5 in 1 month, which will be ample in almost all cases. Under certain conditions it will be found desirable to use another value for the factor of safety.

Example 1.—To find the depth of a slab 8 ft. long that will support a uniform load including its own weight of 400 lbs. per sq. ft. with a factor of safety of 5 in 6 months, using 1:2:4 concrete:

$$d = \sqrt{\frac{l^2 w'}{980}} = \sqrt{\frac{8.0^2 \times 400}{984}} = 5.1 \text{ in.}, \; h = 5.1 \text{ in.} + 9 \text{ in.} = 6 \text{ in.}$$

$$A_s = \frac{d}{100} = \frac{5.1}{100} = .051.$$

For rods ½-in. square, distance center to center

$$= \frac{.25}{.051} = 4.9 \text{ inches.}$$

Example 2.—To find the safe load per square foot on a slab having a clear span of 10 ft. and a depth of 8 ins. from top of slab to center of rods, using a 1:3:6 concrete, with a factor of safety of 5 in 6 months:

$$w' = \frac{888 \times 8^2}{10^2} = 568 \text{ lbs.}$$

$$A = \frac{8}{109} = .0734.$$

For ¾-in. round rods the spacing $= \frac{.4418}{.0734} = 6$ in.

Example 3.—To find the width of a beam having a clear span of 16 ft. and effective depth of 15 ins. that will support a load of 500 lbs. per lin. ft., with a factor of safety of 5 in 6 months, using 1:2:4 concrete:

For 1-in. width,

$$w = \frac{82\, d^2}{l^2} = \frac{82 \times 15^2}{16^2} = 72.07 \text{ ins. wide.}$$

$$\frac{500}{72.07} = 6.94 \text{ ins. wide.}$$

The steel,

$$A_s = \frac{15 \times 6.94}{100} = 1.04 \text{ sq. ins.}$$

Example 4.—To find the breaking load at the center of a beam

12 ft. long, $16\frac{1}{2}$ ins. deep and 6 in. wide, in which d = 15 ins., for a 1:2:4 concrete 1 month old:

$$W = \frac{142.5\ b\,d^2}{l} = \frac{142.5 \times 6 \times 15^2}{12} = 16{,}030 \text{ lbs.}$$

$$\text{Steel} = \frac{6 \times 15}{142} = 0.634 \text{ sq. ins.}$$

Example 5.—To find the safe bending moment M in ft.-lbs. that can be sustained by a beam 22 ins. deep (d = 20 ins.) and 10 ins. wide, factor of safety of 4, concrete 1:3:6, 6 months old:

$$M = \frac{46.25 \times 10 \times 20^2}{4} = 46{,}250 \text{ lbs}$$

The above formulas for the design of beams assume that the strength of the steel in tension is equal to the strength of the

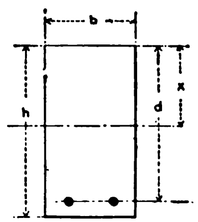


Fig. 200.—Rectangular Beams, Hennebique's Formula.

concrete in compression, as it is believed that this will give the most economic design. The formulas given above both for the design and review of beams may be applied to any mixture or age of concrete or any grade of strength of steel. For work in which great strength is desired, and the saving of weight essential, a 1:2:4 mixture is recommended. Under certain conditions a 1:3:6 mixture may be used.

Hennebique's Formulas for Beams, Slabs and Columns.—The formulas used by Hennebique for computing the strength of beams and slabs are based on the following assumptions: (1) The tensile stresses are carried entirely by the steel. (2) The strain on the concrete is uniform throughout the whole compressive area. (3) The moments of the elastic forces in compression are equal to those of the elastic forces in tension. (Fig. 200.)

Let M = the bending moment in inch pounds;
d = effective depth;
A = area of concrete above center of reinforcement;

then $A = b\,d$;
x = distance of neutral axis below top of beam;
f_c = unit working stress of concrete;
f_s = unit working stress of steel;
A_s = area of steel.

Then $f_c\, x\, b$ = the total compressive stress on the concrete, and
$f_s A_s$ = the total tension on the steel.
Their respective moments about the neutral axis are:

$$\frac{f_c\, b\, x^2}{2} \text{ and } f_s A_s\ (d - x).$$

Each of these quantities is equal to one-half the moment of the external forces, or

$$\frac{f_c\, b\, x^2}{2} = \frac{M}{2}, \text{ and } f_s A_s\ (d - x) = \frac{M}{2}, \text{ or } x = \sqrt{\frac{M}{f_c\, b}} \ldots\ldots (1),$$

which equation gives the position of the neutral axis, and

$$A_s = \frac{M}{2\, f_s\, (d - x)} = \frac{M}{2\, f_s \left(d - \sqrt{\frac{M}{f_c\, b}}\right)} \ldots\ldots\ldots (2).$$

This equation gives the area required for the reinforcement, or if the area be known, the stress in the steel may be obtained by solving for f_s. When this formula is used the depth of the beam, d, is supposed to be known. The last two assumptions on which this theory is founded are evidently inexact.

The value x, it will be seen, is independent of the depth d of the beam. When different depths d are assumed, it will be found that working stresses or factors of safety for the concrete and metal will vary considerably. On this account these formulas do not prove very satisfactory. Hennebique, however, assumes a possible range of working stresses for the concrete, and for the metal, computes the stresses in his beams, and if the resulting stresses fall within the permissible limits, considers them as satisfactory.

Hennebique uses a working stress of 25 kilograms per square centimeter (356 lbs. per sq. in.) for concrete, and from 1,200 to 1,500 kilograms per square centimeter (17,068 to 21,335 lbs.

per sq. in.) for steel. Substituting these values in formulas (1) and (2), viz., $f_c = 25$ kg., and $f_s = 1{,}500$ kg., we then have

$$x = 0.2\sqrt{\frac{M}{b}}, \text{ and } \qquad \text{(3)}.$$

$$A_s = \frac{M}{3{,}000\left(d - 0.2\sqrt{\frac{M}{b}}\right.} \qquad \text{(4)}.$$

Again, assuming a working stress of 350 lbs. per sq. in. for concrete, and 16,000 lbs. for steel, the empirical formulas (3) and (4) become:

$$x = 0.0535\sqrt{\frac{M}{b}} \qquad \text{(5)}.$$

$$A_s = \frac{M}{32{,}000\left(d - .0535\sqrt{\frac{M}{b}}\right.} \qquad \text{(6)}.$$

Example.—Assuming same data as were used for example on page 307, $M = 234{,}375$, assume $d = 14.4$ in. and $b = 12$ ins.:

$$x = .0535\sqrt{\frac{234{,}375}{12}} = 7.48 \text{ ins.}$$

$$A_s = \frac{234{,}375}{32{,}000\,(14.4 - 7.48)} = 1.05 \text{ sq. ins. area of metal required.}$$

Ribbed Slabs.—For ribbed slabs a slight modification of the above formulas is necessary. It is assumed that the area of the slab filling between the beams is added to the compressive area of the beam (Fig. 201).

Let b' = width of slab and g = its thickness. Then, as for beams,

$$\frac{M}{2} = f_c\, b'g\left(x - \frac{g}{2}\right)$$

which expression gives the value $\left(x - \frac{g}{2}\right)$ of the lever arm of the resultant of the compressive forces above an assumed neutral axis. From this assumption we obtain

$$x = \frac{M}{2\, f_c\, b'g} + \frac{g}{2},$$

from which value of x we deduce, d being assumed in the beginning, the value $(d - x)$ of the lever arm for the tensile forces, and there results:

$$A_s = \frac{M}{2\ b'f_s}.$$

It is customary to limit the width of the slab between beams to a value less than 50 times its thickness, i. e., $b' < 50\ g$.

Shear.—Stirrups are employed in Hennebique beams to reinforce the concrete against shearing stresses. The method employed for calculating the stirrups is as follows:

Let S_t represent the maximum shear in the beam. It will

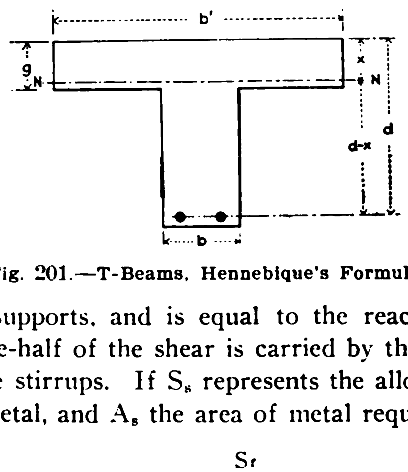


Fig. 201.—T-Beams, Hennebique's Formula.

occur at the supports, and is equal to the reaction. It is assumed that one-half of the shear is carried by the bent bars and one-half by the stirrups. If S_s represents the allowable shearing stress in the metal, and A_s the area of metal required,

$$A_s = \frac{S_t}{2\ S_s}.$$

This formula gives the total section of the stirrups required in a length of the beam equal to the distance between the center of compression and the center of tension. This distance is assumed to be equal to $d - \frac{g}{2}$. See Fig. 201 for ribbed slabs and for slabs and rectangular beams, $d - \frac{x}{2}$. When the spacing z of the first two stirrups at the end of the beam differ from this spacing their section is modified to correspond to the spacing used. Furthermore, when a number of stirrups are placed in the same transverse section of a beam, it is considered that each stirrup is composed of two branches, and if there be n stirrups the total section for each branch will be

$$\frac{A_s}{2} = \frac{S_f}{4\, S_s\, n} \times \frac{z}{d - \frac{g}{2}}.$$

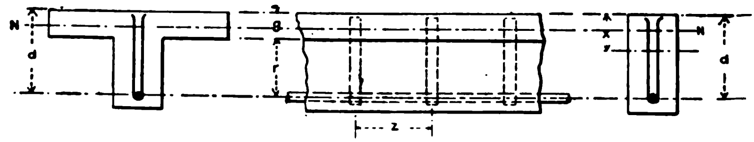


Fig. 202.—Stirrup Diagram, Hennebique's Formula.

The following analysis will make clear the development of this theory for stirrups.

From Fig. 202:

$$\frac{S_f}{2}\, z = A_s\, S_s\, r,$$

and

$$A_s = \frac{S_f\, z}{2\, S_s\, r};$$

but r is taken equal to $d - \frac{g}{2}$ and we have for the total section of a stirrup, i. e., its two branches:

$$A_s = \frac{S_f\, z}{2\, S_s \left(d - \frac{g}{2}\right)}.$$

and for a rectangular beam

$$A_s = \frac{S_f\, z}{2\, S_s \left(d - \frac{x}{2}\right)}.$$

Hennebique allows a shearing stress of 800 kilograms per sq. cm. (11,380 lbs. per sq. in.) for steel.

In computing the maximum bending moment at the center for beams under a uniform load w per lineal foot or meter, Hennebique uses the usual formula, $M = \frac{1}{8}\, w\, l^2$ for principal beams or girders, but for secondary beams which are more or less continuous over the principal beams he employs the formula $M = {}^1/_{10}\, w\, l^2$. In figuring rectangular floor slabs which are approximately square and supported at all four edges, he assumes them to be fixed at the edges, and uses the formula $M = {}^1/_{36}\ w\ l^2$ to obtain the required maximum bending moment.

Columns.—To determine the strength of columns and walls, Hennebique considers that both the concrete and the metal may be strained up to their maximum working value in compression at the same time and simply puts

$$P = f_s A_s + f_c A_c.$$

The working values assumed for steel and concrete in compression are respectively $f_s = 1{,}000$ kg. per sq. cm. (14,200 lbs. per sq. in.) and $f_c = .25$ kg. per sq. cm. (350 lbs. per sq. in.).

This formula is inexact as regards the true internal stress in the two materials, for as f_c and f_s are assumed at 25 kg. and 1,000 kg. per sq. cm., we have

$$e = \frac{E_s}{E_c} = \frac{A_s}{A_c} = \frac{1{,}000}{25} = 40,$$

a much higher value for e than is warranted. A higher stress than 25 kg. will result in the concrete if the steel is strained up to 1,000 kg., and the highest stresses will result in the concrete when high percentages of reinforcement are used.

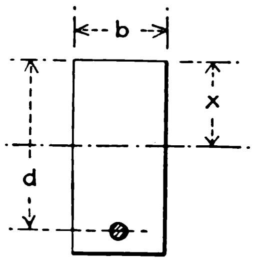


Fig. 203.—Diagram, Coignet's Formula.

Coignet's Formula.—In designing pipes, Coignet uses the following formula:

$$A_s = \frac{p\,d}{2\,f_s},$$

in which p = pressure in pounds per sq. inch or kilograms per sq. cm., d = internal diameter of pipe, and f_s the tensile working stress in lbs. per sq. inch, or kilograms per sq. cm. of the metal. To calculate the longitudinal rods, Coignet considers the section of concrete included between the adjacent coils as being a slab fixed at its extremities.

Coignet employs the following formulas for computing slabs (Fig. 203):

$$x = \frac{8}{5} \frac{A_s f_s}{f_c b}.$$

Assuming $f_s = 1{,}500$ kg. and $f_c = 40$ kg., $x = 60 A_s$, b being equal to unity, or $bx = 60 A_s$.

This engineer also uses the following empirical formula:

$A_s = \frac{2}{3}$ d.
$x = \frac{2}{5}$ d.
$M = 6.4 d^2$.

Bonna's Formula.—M. Bonna uses reinforcing rods in both the tension and compression flanges of beams. His formulas are as follows: If A_s represents the sectional area of the tension bars calculated for the total load, $2/3\ A_s$ is used for the section of the upper or compression bars. For a simple beam, $M = 1{,}500\ d\ A_s$, as he assumes $f_s = 1{,}500$ kg., and

$$A_s = \frac{M}{1{,}500\ d'},$$

d' being the distance between the center of gravity of the two reinforcements. A_s' the section of the upper reinforcement then is

$$A_s' = \frac{2}{3} A_s = \frac{M}{2.250\ d'}.$$

If f_s be assumed at 15,000 lbs. per sq. in., the above formulas become

$$A_s = \frac{M}{15{,}000\ d'}, \text{ and } A_s' = \frac{M}{22{,}500\ d'}.$$

This method of computation avoids the necessity of determining the position of the neutral axis, which is always a tedious solution to make.

Johnson's Theory.—The following theory for reinforced concrete beams has been developed by Mr. A. L. Johnson, M. Am. Soc. C. E., Consult. Engr. for the Expanded Metal and Corrugated Bar Co. It is based on the following assumptions: That plain sections before bending are plane after bending, up to the elastic limit of the metal and up to the full compressive strength of the concrete. It is also assumed that such a quantity of metal is used as will cause the elastic limit in the reinforcement and the compressive strength of the concrete to be reached at the same time. It is assumed that the modulus of elasticity of steel is constant up to the development of its elastic limit. When the elastic limit of the steel has been reached, the construction as a whole has reached its maximum carrying capacity, for beyond this point the elongation of the steel will be so great that the concrete will be ruptured, hence a steel with a high elastic limit is desirable in this kind of construction. As regards the concrete its maximum strength and load carrying capacity will occur when the extreme fibre stress on the concrete becomes equal to the compressive stress, assuming a metal section of sufficient strength is used to develop it. When this occurs, referring to Fig. 204, we assume that the shaded area above the neutral axis represents the complete compressive stress diagram

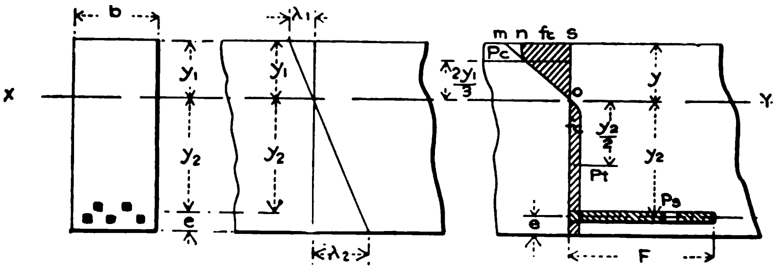


Fig. 204.—Stress-Strain Diagram for Johnson's Formula.

of the concrete, o s being the axis of proportionate elongation, and the neutral axis the axis of stress per square inch. Mr. Johnson states he has found from an examination of a great many such diagrams, that the resultant modulus, represented by the tangent of the angle n o s, is about two-thirds in rock concrete and one-half in cinder concrete, as much as the original modulus, represented by the tangent of the angle m o s.

Also that the total area for both kinds of concrete is about one-quarter larger than the triangular area n o s. These assumptions seem crude at first, but as a matter of fact, they are not more so than would be any formula intended to represent the compressive stress diagram for a class of concrete. The latter would give all points on the curve, whereas our method gives only the end of the same; but our location of that point is as accurate as can be obtained by any method. The development of the formulas for rectangular beams follows:

Figure 204 shows a cross section of a steel-concrete beam; strain diagram at the ultimate load, the stress diagram corresponding to the above strain diagram.

Let E_s = Modulus of elasticity of steel in pounds per square inch.
E_c = Modulus of elasticity of the concrete in compression in pounds per square inch.
F = Elastic limit of steel in pounds per square inch.
f_c = Compressive strength of concrete in pounds per square inch.
f_t = Tensile strength of concrete in pounds per square inch.
b = Width of section in inches.
a^2 = Area of one bar in square inches.
d = Spacing of bars in inches.
$\frac{a^2}{d}$ = Number of square inches of metal per inch of width.
$\frac{a^2b}{d}$ = Total quantity of metal in width b.
M_o = Moment of ultimate resistance of cross-section in inch pounds.
M = Bending moment of external forces in inch pounds.
W = Total load on beam in pounds.
P_s = Total stress on metal in width b in pounds.
P_c = Total comprehensive stress on concrete in width b.
P_t = Total tensile stress in concrete in width b.
λ_1 = Unit elongation of extreme fiber in compression.
λ_2 = Unit elongation of steel.
e = Distance in inches from extreme fiber on tension side to middle plane of metal reinforcement. This thickness is not figured into the strength of the beam.

We can then write the following equations:

$$P_c = \tfrac{5}{8} f_c b y_1 \quad \ldots\ldots (1)$$

But

$$f_c = \frac{2\ E_c \lambda_1}{3}$$

And

$$\lambda_1 = \lambda_2 \frac{y_1}{y_2}$$

Then

$$\lambda_2 = \frac{F}{E_s}$$

And

$$\lambda_1 = \frac{F y_1}{E_s\ y_2}$$

Or

$$f_c = \frac{2\ F E_c y_1}{3\ E_s\ y_2}$$

$$y_2 = \frac{2\ F E_c}{3\ f_c\ E_s} y_1 \quad \ldots\ldots (2)$$

For the steel,

$$P_s = \frac{Fa^2b}{d} \qquad \text{........................(3)}$$

For the concrete in tension,

$$P_t = {}^8/_{10}\, f_t b y_2 = \frac{8\, FE_c f_t b y_1}{15\, f_c E_s} \qquad \text{..................(4)}$$

The empirical constant 8/10 is derived from the results of M. Considère.

We then have,

$$P_c = P_s + P_t \qquad \text{........................(5)}$$

Or,

$$\frac{5\, f_c b y_1}{8} = \frac{Fa^2b}{d} + \frac{8\, FE_c f_t b y_1}{15\, f_c E_s} \qquad \text{..............(6)}$$

From which

$$\frac{a^2b}{d} = \frac{75\, f_c b_1 - 64\, f_t b y_1 \left(\dfrac{FE_c}{f_c E_s}\right)}{120\, F} \qquad \text{..............(7)}$$

For the moment of resistance we have,

$$M_o = \frac{Fa^2b}{d}\left(y_2 + \frac{2\, y_1}{3}\right) + \frac{8\, f_t b y_2}{10}\left(\frac{y_2}{2} + \frac{2\, y_1}{3}\right) \qquad \text{............(8)}$$

The size of beam needed to develop a required moment of resistance can now be readily obtained from equations (2), (7) and (8). From (2) we obtain a numerical ratio between y_1 and y_2 when the constants depending only upon the particular materials used are known. Equation (7) gives the quantity of metal required in terms of y_1, all other factors being known constants for the given materials. Then (8) gives the value of the ultimate moment of resistance in terms of y_1 only. As the moment of resistance is to equal the bending moment of the external proof loads, M_o in equation (8) is known, which at once gives the value of y_1, from which all other values may be determined.

We have found the best average values for the constants for 1:3:6 rock concrete to be the following:

$$E_c = 3{,}000{,}000,\ f_c = 2{,}000, \text{ and } f_t = 200.$$

For the steel the value of E_s varies but little for the different grades of rolled material, but F, or the elastic limit, varies greatly. As before stated in the introduction, we can not utilize any of the strength of the steel beyond the elastic limit, therefore it is desirable that this limit should be fairly high.

Our corrugated bars have an elastic limit of between 50,000 and 60,000 pounds per square inch. We therefore use for the constants for the steel,

$$E_s = 29{,}000{,}000, \text{ and } F = 50{,}000.$$

With these values equations (2), (7) and (8) reduce to the following respectively:

$$\left.\begin{array}{l} y_2 = 1.72y_1 \\ \dfrac{a^2b}{d} = .0195by_1 \\ M_o = 2750by_1^2 \\ h = y_1 + y_2 + e \end{array}\right\} \begin{array}{l} \text{If } b = 12 \text{ in.} \\ \text{and } e = \dfrac{h}{10} \\ \text{we have,} \end{array} \left\{\begin{array}{l} y_1 = .368d \ldots\ldots\ldots\ldots (9) \\ \dfrac{a^2b}{d} = 0.077h = .64\% \ldots (10) \\ M_o = 3620h^2 \ldots\ldots\ldots\ldots (11) \end{array}\right.$$

There are certain grades of rock that give a much more compressible concrete than the above and have at the same time a greater compressive strength. Trap rock falls within this category as well as certain kinds of western limestone using a well proportioned aggregate and a mix of 1 : 2 : 5. For such concrete we may assume the following constants:

$$E_c = 2{,}400{,}000, \; f_c = 2{,}400, \; f_t = 200.$$

Using the same values for the steel our equations of design then become:

$$\left.\begin{array}{l} y_2 = 1.15y_1 \\ \dfrac{a^2b}{d} = .0263by_1 \\ M_o = 2620by_1^2 \\ h = y_1 + y_2 + e \end{array}\right\} \begin{array}{l} \text{If } b = 12 \text{ in.} \\ \text{and } e = \dfrac{h}{10} \\ \text{we have,} \end{array} \left\{\begin{array}{l} y_1 = .418h \ldots\ldots\ldots\ldots (12) \\ \dfrac{a^2b}{d} = 0.132h = 1.1\% \ldots (13) \\ M_o = 5505h^2 \ldots\ldots\ldots\ldots (14) \end{array}\right.$$

For a 1 : 2 : 5 mix of cinder concrete we have

$$E_c = 750{,}000, \; f_c = 750, \text{ and } f_t = 80.$$

For this material the equations become:

$$\left.\begin{array}{l} y_2 = 0.862y_1 \\ \dfrac{a^2b}{d} = .00827by_1 \\ M_o = 693by_1^2 \\ h = y_1 + y_2 + e \end{array}\right\} \begin{array}{l} \text{If } \quad = 12 \text{ in.} \\ \text{and } e = \dfrac{h}{10} \\ \text{we have,} \end{array} \left\{\begin{array}{l} y_1 = .483h \ldots\ldots\ldots\ldots (15) \\ \dfrac{a^2b}{d} = 0.048h = .4\% \ldots (16) \\ M_o = 1935h^2 \ldots\ldots\ldots\ldots (17) \end{array}\right.$$

In beams of Tee section (Fig. 205) y_1 is the same as for rectangular sections inasmuch as the position of the neutral axis is determined by the relative values of maximum compressibility

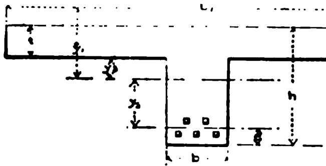


Fig. 205.—T-Beam Diagram for Johnson's Formula.

of the concrete and extensibility of the steel inside the elastic limit or by the ratio of λ_1 and λ_2. This is, of course, only true at the maximum load.

We then have as before,

$$y_2 = \frac{2\,FE_c}{3\,f_cE_s}y_1 \quad \text{.......................(18)}$$

VALUES OF b_1 AND t.

Let S_v = Total shear in pounds along the two vertical planes of attachment between the wings and beam;

S_h = Total shear in pounds along the horizontal plane of attachment between the rib and floor plate;

s = Maximum shearing strength of concrete in pounds per square inch;

$K = \dfrac{y_2}{y_1}$

l = Length of span in feet;

P_c' = Total compression in pounds at maximum load between neutral axis and underside of floor plate;

P_c'' = Total compression in pounds in flange at maximum load.

All other functions are as shown by Fig. 205, and are in inches.

There are three methods of failure above the neutral axis:

1. By compression in the flange;
2. By deficiency in S_v owing to smallness of t;
3. By deficiency in S_h owing to smallness of b.

It would be desirable to have equal strength in all these directions, but this is not always possible owing to other considerations. Where it is possible we have,

$P_c'' = S_v = S_h$ ....(19)

But $S_h = 3\,bsl$ ....(20)

and $S_v = 6\,tsl$ ....(21)

The shearing stress is a maximum at the ends and for uniformly loaded beam varies uniformly to zero at the center. The value S_v may be increased about 50 per cent., owing to the metal reinforcement in the underside of floor plate, which is always present in these designs. If vertical shear bars were used the same increase could be made in S_h, but ordinarily these would not be used so we will not separately discuss this condition. Equation (21) then becomes

$$S_v = 9\,tsl \qquad (22)$$

Assuming the compression stress diagram to be a parabola

$$P_c'' = \tfrac{2}{3}\,(1 - K^{3/2})\; f_c\, b_1\, y_1 \qquad (23)$$

This is on the assumption that the outer ends of the wings would be just as heavily stressed as the portion next to the beam. This would not be the case, the stress varying according to the ordinates to a parabola from zero at the outer ends to a maximum at the beam, and we should, therefore, multiply the above value by 2/3. The portion of this width over the beam itself would not be subject to this modification, but there are other influences tending to offset this so that the above is sufficiently correct.

Then $$P_c'' = \tfrac{4}{9}\,(1 - K^{3/2})\; f_c b_1 y_1 \qquad (24)$$

From (20) and (22) we see that if t is not less than $\frac{b}{3}$, failure will not occur along the vertical sides of beam where wings attach. Now we will assume at once that t will not be allowed to have a value less than this. This leaves us to consider the relation between P_c'' and S_h only. We then have from (20) and (24)

$$3\,bsl = \tfrac{4}{9}\,(1 - K^{3/2})\; f_c b_1 y_1,$$

from which

$$b_1 = \frac{27\,bsl}{4\,(1 - K^{3/2})\; f_c y_1} \qquad (25)$$

The theoretical relation between s and f_c is

$$s = \frac{f_c}{2 \tan \theta}$$ (see Johnson's Materials of Construction, p. 29). (26)

where θ is the angle made by the plane of rupture on a compression specimen of moderate length with a plane at right angles to the direction of stress.

For concrete this angle is about 60°, hence

$$s = \frac{f_c}{3.464} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (27)$$

But this value is high in view of the liability of concrete to crack and we recommend that twice the strength be provided in the shearing values on this basis that is used in compression.

We would then have $S_h = 2 P_c''$, or

$$3 \, bsl = {}^8/_9 \, (1 - K^{3/2}) \, f_c b_1 y_1$$

from which

$$b_1 = \frac{27 \, bsl}{8 \, (1 - K^{3/2} \, f_c y_1}$$

and substituting the value of s we have with sufficient accuracy,

$$b_1 = \frac{bl}{(1 - K^{3/2}) \, y_1} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (28)$$

We will now insert this value in (24) and proceed to obtain the moment of resistance. At times the above value of b_1 would be greater than the spacing of the beams, in which case the latter distance would be used for the value of b_1 in (24), and the other values worked over on this basis.

From (24) and (28) then we have,

$$P_c'' = {}^4/_9 \, f_c bl \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (29)$$

also

$$P_c' = {}^2/_3 \, K^{3/2} \, f_c b y_1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (30)$$

Then

$$P_c = \frac{2 \, f_c b}{3} \left(\frac{2l}{3} + K^{3/2} \, y_1 \right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (31)$$

$$P_t = .8 \, f_t b y_2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (32)$$

$$P_s = \frac{Fa^2 b}{d} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (33)$$

But

$$P_c = P_t + P_s \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (34)$$

From which

$$\frac{a^2 b}{d} = \frac{1}{F} \left[\frac{2 \, f_c b}{3} \left(\frac{2l}{3} + K^{3/2} \, y_1 \right) - .8 \, f_t b y_2 \right] \ldots\ldots (35)$$

and

$$M_o = P'_c \frac{K y_1}{2} + P''_c \frac{(1 + K) y_1}{2} + P_t \frac{y_2}{2} + P_s y_2 \ldots . (36)$$

Problem: Required the size of Tee shaped beam necessary to carry a total ultimate load of 600 pounds per square foot on a span of thirty-two feet, ribs to be nine feet apart.

Then

$$M = \frac{12 \times 9 \times 600 \times 1{,}024}{8} = 8{,}300{,}000 \text{ inch pounds.}$$

Let us assume a depth of beam h equal to 22 in. Then $y_1 + y_2 = 20$ in. For this spacing of beams the thickness of floor plate should be 4 in.

Using special rock concrete we have from (12)

$$y_1 = \frac{20}{2.15} = 9.3 \text{ in. and } y_2 = 10.7 \text{ in.}$$

$$K = \frac{y_1 - t}{y_1} = \frac{5.3}{9.3} = .57, \text{ and } K^{3/2} = .43.$$

Then

$$P_c' = \tfrac{2}{3} K^{3/2} f_c b y_1 = \tfrac{2}{3} \times .43 \times 2{,}400 \times 9.3b = 6{,}400 \text{ b}$$

$$P_c' = {}^4/_9 f_c b l = {}^4/_9 \times 2{,}400 \times 32b = 34{,}100 \text{ b}$$

and

$$P_c = 40{,}500 \text{ b}$$

$$P_t = .8 f_t b y_2 = .8 \times 200 \times 10.7b = 1{,}715 \text{ b}$$

Then

$$P_s = 38{,}785 \text{ b}$$

and

$$\frac{a^2 b}{d} = \frac{38{,}785 \text{ b}}{50{,}000} = .776 \text{ b}$$

$$M_o = P'_c \frac{Ky_1}{2} + P''_c \frac{(1 + K)y_1}{2} + P_t \frac{y_2}{2} + P_s y_2$$

$$= 6{,}400 \text{ b} \times 2.65 + 34{,}100 \text{ b} \times 7.3 + 1{,}715 \text{ b} \times 5.35 + 38{,}785 \text{ b} \times 10.7$$

$$= 690{,}130\text{b}$$

or,

$$b = \frac{8{,}300{,}000}{690{,}130} = 12.03 \text{ in.}$$

Substituting in (28) we have

$$b_1 = \frac{12 \times 32}{.57 \times 9.3} = 72.5 \text{ in. or 6 ft.}$$

As this value of b_1, which we have used in determining the value of P_c'' above, is less than the spacing of the beams it is the proper one to have used. It will be noted that t is just one-third of b.

From the foregoing we derive the following relations for good grade of 1 : 2 : 5 Portland cement rock, concrete, where f_c = 2400; $f_t = 200$; $E_c = 2{,}400{,}000$; $E_s = 29{,}000{,}000$; $F = 50{,}000$.

$$P'_c = 1{,}600\ K^{2/3}\ by_1;\quad P_t = 160\ by_2;\quad P''_c = 1{,}066\ bl.$$

Also $\dfrac{a^2b}{d} = \dfrac{P'_c - P_t + P''_c}{50{,}000}$ = number of square inches of metal required in rib.

$$M_o = P_c'\left(\frac{y_1}{2} + y_2 - \frac{t}{2}\right) + P_c''\left(y_1 + y_2 - \frac{t}{2}\right) - P_t\frac{y_2}{2} =$$

ultimate moment of resistance in inch pounds.

All measures of length in inches except l, the length of span, which is in feet.

The value of t must not be less than one-third b.

The value of b represents the maximum width of flange that can be utilized in figuring the strength of the Tee, and its value is:

$$b_1 = \frac{bl}{(1 - K^{2/3})y_1}.$$

Where this value of b_1 exceeds materially the distance between the ribs, the above formulæ can not be used, and the value of P_c'' would have to be obtained from the general equation (24).

SHEAR IN STEEL-CONCRETE BEAMS.

Let M_o' = moment of resistance in inch pounds at 12 in. from end of beam carrying its ultimate load.
M_o = ultimate moment of resistance in inch pounds at center.
l = span of beam in feet.
λ_2 = elongation per inch at the plane of the metal, at section 12 in. from end.
b = width of beam in inches.
s = ultimate shearing strength of the concrete, about one-fourth the ultimate compressive strength.
Other functions as already given.

Then $M_o' = \dfrac{4l - 4}{l^2} M_o$ for uniformly loaded beam..................(1)

$$\lambda_2 = \frac{M_o'}{\dfrac{E_c\,by_1^3}{3\,y_2} + \dfrac{E_c\,by_2^2}{3} + \dfrac{E_s\,a^2by^2}{d}} \quad ....(2)$$

$$by_1^2 = by_2^2 + \frac{2\,E_s\,a^2b}{E_c\,d}\,y_2 \quad ....(3)$$

$$y_1 = h - y^2 - e \quad (4)$$

$$P_s = \frac{E_s \lambda_2 a^2 b}{d} \quad (5)$$

After designing the beam by the beam formula given above, $\frac{a^2b}{d}$, $y_1 + y_2$, E_c, E_s, and b are known. From (1) we obtain M'_s and from (3) and (4), y_1 and y_2. From (2) will be obtained λ_2, which inserted in (5) will give the pull in the bars which has to be absorbed by shearing stress in the concrete over an area = 12 b. As it is desirable to take twice the factor of safety in shear that is taken in bending, P_s should not exceed 6 b s, where s is taken at one-fourth the compressive strength of the concrete.

If beams are loaded at two points some distance apart the maximum shearing stress is likely to be a very different character. The bending moment being uniform between the loading points, the first cracks on the tension flange are as apt to occur under one of the loads as in the middle and this will greatly reduce the

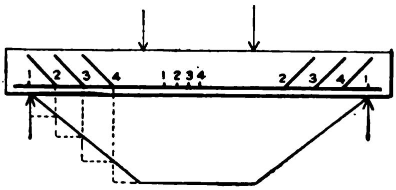


Fig. 206.—Shear Diagram, Johnson's Formula.

strength of the anchorage of the ends of the bars represented by the shearing resistance of the concrete along the plane just above the metal between the crack and the end of the beam. This is especially true as the maximum shearing stress along this plane is likely to be double the average stress. In such cases, as also in cases of uniform load where the shear exceeds the limits above given, the bars should be bent up at the ends, as shown in Figs. 206 and 207.

The discussion of formulas 1 to 17, only applies to beams on knife-edge supports and is inadequate when applied to rectangular beams used in flooring where there are haunches on the beams and flooring in the adjacent panels. There is no scientific discussion of such a construction, but we know from experience that it has about twice the carrying capacity of a construction acting as a simple beam resting on knife-edge supports. The haunches on the beams produce a continuous girder action, such that the external bending moment in the middle is only about two-thirds as great as if it were free at the ends; also the floor in the adjacent panels produces an interior arching action within the thickness of the slab, increasing the area of the compressive stress diagram, about one-third of this extra compression being counteracted by the adjacent floor panels, instead of by the reinforcement. The effect of these two elements is to double the

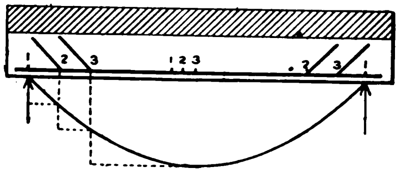


Fig. 207.—Shear Diagram, Johnson's Formula.

strength of the beams as used in floors. If the beam does not have haunches projecting below, but is itself the full depth throughout, then only one-third should be added to the value of the moment of resistance.

Beams of Tee shape are not greatly strengthened by incorporation in floor panels, inasmuch as most of the compressive strength comes from the flanges too high up to be affected by the interior arching action. That is to say, P''_c would remain practically the same and P'_c would be increased probably fifty per cent. But the latter is usually so small as to make this increase of little value.

CHAPTER XXI.

THEORY OF COLUMNS.

Reinforced concrete columns are of two general types: First columns with straight reinforcing rods, and, second, hooped columns.

Concrete Columns with Straight Reinforcing Rods.—These may be divided into two classes. First: When light reinforcement is used. In this class may be included all columns where the area of the reinforcement is only a small percentage of the cross-section of the column and no appreciable error will result if it be neglected when computing the sectional area of the concrete. Second: When a higher percentage of metal is used and it becomes necessary to deduct its area from the total area of the piece to obtain the correct area of the concrete.

We will assume a column, having a cross-section of any form, round, square, rectangular, etc., and reinforced with straight bars of round, square or any desired section, subjected to compression due to a load P, acting along its axis. If the column was composed of homogeneous materials the sectional area required to sustain the load P would be

$$P = f_c A + f_s^c A_s \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

Where f_c = the allowable working stress on the concrete and f_s^c that of the metal, A, represents the sectional area of the concrete and A_s that of the metal. The deformation due to a load P will cause the displacement of any section taken at right angles to the axis of the column, to a position parallel to itself. In order that the concrete and the metal may act as a homogeneous material their deformations must be equal, and we have the condition that the stress upon each material must be proportioned to its coefficient of elasticity, or

$$\frac{f_c}{E_c} = \frac{f_s^c}{E_s} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

when E_c and E_s are the coefficients of elasticity of the concrete and the metal respectively. Equation (2) may be written

$$f_s^c = f_c \left(\frac{E_s}{E_c}\right) \qquad (3)$$

Substituting this value of F_s in equation (1), there results

$$P = f_c A + f_c \left(\frac{E_s}{E_c}\right) A_s$$

$$= f_c \left[A + \left(\frac{E_s}{E_c}\right) A_s \right] \qquad (4)$$

The ratio of the coefficient of elasticity of steel to that of concrete is usually expressed by $e = \frac{E_s}{E_c}$

and formulas (3) and (4) become

$$f_s^c = e f_c \qquad (5)$$

$$P = f_c (A + e A_s) \qquad (6)$$

Columns Heavily Reinforced.—When the percentage of metal used is increased it is necessary to determine more closely the sectional area of the concrete. Subtracting from the total sectional area the area of the metal the true area of the concrete is obtained, or $A_c = A - A_s$.

Substituting this value in equation (6) and reducing

$$P = f_c (A + (e - 1) A_s) \qquad (7)$$

This equation may be used for computations of columns in which the percentage of metals used is greater than 1 per cent.

The above formulas are applicable to columns having a length not exceeding from 20 to 25 times their diameter. When a greater length than this is used, Euler's formula for flexure, given below, should be used. For all practical purposes it will be found that no formula for flexure will be needed, as columns longer than 25 diameters are seldom used. For important work and very heavy loads it will be well to limit the length to 20 diameters or less. In classic architecture the total length of the column, including the base and capitol, does not exceed from 10 to 15 diameters. Slender columns have an appearance of weakness, and for this, if for no other reason, should often be avoided. Again, great difficulty will be experienced in securing a thoroughly homogeneous concrete when constructing slender columns. It will often be advisable on the score of economy to use larger sections than theoretical requirements call for.

Euler's formula for long columns with fixed ends is (see Merriman's Mechanics of Materials):

$$P = 4\frac{\pi^2 EI}{l^4} \quad \ldots\ldots (8)$$

in which

I = the moment of inertia of the column.

E = the coefficient of elasticity and may in the case of reinforced concrete columns be taken as E_c, the coefficient of elasticity of the concrete.

l = the length of the column in inches.

For a rectangular column of reinforced concrete

$$I = {}^1/_{12}\, bd^3 + e A_s y^2 \quad \ldots\ldots (9)$$

y being the distance of the center of the reinforcing bars from the axial plane of the column.

For square ended columns formula (8) becomes

$$P = \frac{4\pi^2\left(\dfrac{bd^3}{12} + e A_s y^2\right) E_c}{l^2} \quad \ldots\ldots (10)$$

Substituting for e, a value of 10 and for π^2 its value, this formula becomes

$$P = \frac{39.48\left(\dfrac{bd^3}{12} + 10 A_s y^2\right) E_c}{l^2} \quad \ldots\ldots (11)$$

Formula (11) gives the ultimate strength of the column, and to obtain a working formula a suitable factor of safety should be used. For square columns b = d and the expression

$$\frac{bd^3}{12} \text{ becomes } \frac{d^4}{12},$$

and equation (11) becomes

$$P = 39.48\frac{\left(\dfrac{d^4}{12} + 10 A_s y^2\right) E_c}{l^2} \quad \ldots\ldots (12)$$

For circular columns and columns of polygonal section approximating a circular form, the value of the moment of inertia I when d = the diameter of the columns, becomes

$$I = (0.0491\, d^4 + 10\, A_s y^2) \quad \ldots\ldots (13)$$

Substituting this value of I in equation (8) it becomes

$$P = \frac{4\pi^2\ (0.0491\ d^4\ +\ c\ A_s\ y^2)\ E_c}{l^2} \quad \ldots\ldots\ldots\ldots\ldots (14)$$

replacing e and π by their numerical values, we obtain for the ultimate strength of round columns

$$P = 39.48\ \frac{(.0491\ d^4 + 10\ A_s\ y^2)\ E_c}{l^2} \ldots\ldots\ldots\ldots\ldots (15)$$

For heavily reinforced sections Euler's formula for columns with fixed ends takes the form

$$P = \frac{4\pi^2\,[(I_c - I_s) + e\,I_s]\ E_c}{l^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (16)$$

When I_c = the moment of inertia of the whole section and I_s = the least moment of inertia of the reinforcement with reference to the axial plane of the column and E_c, as before, the coefficient of elasticity of the concrete.

Now, $I_c = A_s\ y^2$ when A_s = the area of the metal and y is its distance from the axial plane. I_c has the same value for rectangular, square and round sections as before, and the equation becomes:

For rectangular columns

$$P = \frac{4\pi^2\left(\frac{bd^3}{12} + (e - 1)\ A_s\ y^2\right) E_c}{l^2} \quad \ldots\ldots\ldots\ldots\ldots (17)$$

For square columns

$$P = \frac{4\pi^2\left(\frac{d^4}{12} + (e - 1)\ A_s\ y^2\right) E_c}{l^2} \quad \ldots\ldots\ldots\ldots\ldots (18)$$

and for round columns

$$P = \frac{4\pi^2\,[.0491\ d^4 + (e - 1)\ A_s\ y^2]\ E_c}{l^2} \quad \ldots\ldots\ldots\ldots (19)$$

Substituting the values of π and e, as before, formulas (17) (18) and (19) become, respectively:

For rectangular columns

$$P = \frac{39.48\ \left.\frac{bd^3}{12} + 9\ A_s\ y^2\right]\ E_c}{l^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (20)$$

For square columns

$$P = \frac{39.48\left[\frac{d^4}{12} + 9 A_s y^2\right] E_c}{l^2} \quad \text{................(21)}$$

For round columns

$$P = \frac{39.48\, [.0491\, d^4 + 9 A_s y^2]\, E_c}{l^2} \quad \text{..............(22)}$$

It should be remembered that formulas (20), (21) and (22) are for the ultimate strength, and when a working formula is desired a proper factor of safety should be introduced. When it is desired to use another value for e than 10, it should be introduced in equations (6), (7), (10), (11), (15), (16), (17), (18), and (19).

Formulas 20, 21 and 22 may be used to check the columns of existing structures or to design a column to support a given load. When used to check a column, the sectional area of both the concrete and the metal, together with the load, may be given to determine the working stress, f_c, in the concrete, or safe working stresses may be assumed and a value for P computed. When the design of a column to support a given load, P, is to be made, a working value for f_c is assumed; we have then to determine the value of A and A_s and the problem becomes a tentative one. Usually practical considerations fix within narrow limits the size of column desired and the area of steel necessary to supplement the concrete or the percentage of steel to be used is fixed upon in advance, and the areas of concrete and steel may be determined by assuming a section and computing its strength by the formula to determine if the working stresses f_c and f_s' approximate the working stress determined upon for the given structure. Usually a section having the proper working stresses may be found after two or three trials. Examples will be given showing the use of the formulas, both for checking and designing columns.

Example 1.—Let it be required to check the section of the Hennebique column described on page 467.

The section of the column is 35 × 35 cm. = 1225 sq. cm. = total area of col. 4 rods 20 mm. diam. = 3.14159 × 10² × 4 = 1257 sq. mm. = 12.57 sq. cm. area of metal. Comparing the total area of reinforcement with the area of column it is observed that the reinforcement is approximately 1 per cent. We will, therefore, use formula (6),

$$P = f_c \, (A + 10 \, A_s).$$

The total load was 43,000 kg., and we have

$$f_c = \frac{43{,}000}{1{,}225 + 10 \times 12.57} = 31.8 \text{ kg.}$$

total compression per sq. cm. on concrete, or approximately 450 lbs. per sq. in., which, while a little high for the load on column, is at times used by some designers.

Example 2.—Determine the safe load which can be carried by the column for a Chicago store building, shown in Fig. 319. The size of the column is 20 × 20 ins. = 400 sq. in. 4 rods 2 $\frac{9}{16}$ ins. diameter = 5.1572 × 4 = 20.6288 sq. ins., or approximately 5 per cent. In this case we will use formula (7). $P = f_c \, (A + (e - 1) \, A_s)$. It is necessary to assume a safe working stress for f_c. The concrete used was a 1:2:2 mixture, and its crushing strength may be taken at 3,600 lbs. If a factor of safety of 6 be used, the working strength will be 600 lbs. per sq. in., and we have, substituting in the formula, taking e = 10,

$$\begin{aligned} P &= 600 \, (400 + 9 \times 20.63) \\ &= 351{,}400 = 175.7 \text{ tons.} \end{aligned}$$

Example 3.—Let it be required to determine the size of a column 12 ft. long necessary to carry a load of 40 tons, using a 1:2:4 concrete with a working stress of 500 lbs. per sq. in., reinforced with 3 per cent. of metal. We will assume that e has a value of 10.

We then have

$$P = 40 \text{ tons} = 80{,}000 \text{ lbs.}$$

$$f_c = 500, \; e = 10, \; p = .03, \text{ and } A_s = .03\,A;$$

and Formula (7) is

$$P = f_c \, [A + (e - 1) \, A_s].$$

Transposing,

$$\frac{P}{f_c} = A + 9 \times .03\,A = 1.27\,A.$$

$$\frac{80{,}000}{500} = 1.27\,A.$$

	A = 126	sq. ins.
Add 2 sq. ins. for chamfers	2	" "
A_s = 126 × .03 =	3.78	" "
	131.78	sq. ins.

A column 11.5 × 11.5 ins. = 132.5 sq. in. 4 rods $1\frac{1}{8}$ ins. in diameter = 0.994 × 4 = 3.976 sq. in., giving an area of reinforcement slightly in excess of area required.

As the column required is only 12 ft. long, it will not be necessary to test it by Euler's formula for bending.

Example 4.—What will be the size of column necessary to carry the same load as in the last example, but reinforced with 1 per cent. of metal.

As before

$$P = 40 \text{ tons} = 80{,}000 \text{ lbs.}$$
$$f_c = 500 \text{ lbs.} \qquad e = 10.$$
$$p = .01, \text{ and } A_s = .01\ A.$$

As only 1 per cent. of reinforcement is to be used, we will employ Formula 6.

$$P = f_c\ (A + e\,A_s)$$

or,

$$\frac{80{,}000}{500} = 1.10\ A, \text{ and}$$

$$A = 145.45 + \text{sq. ins.}$$
$$A_s = 1.45 + \text{sq. ins.}$$

A column $12\frac{1}{4} \times 12\frac{1}{4}$	= 150	sq. ins. concrete.
Less 2 sq. ins. for chamfers	2	" " "
	148	sq. ins. concrete.

Four rods $^{11}/_{16}$ in. diameter = 1.48 sq. ins. metal. These sections will be adopted. The column with 1 per cent. reinforcement will be the more economic of the two. Thus, assuming the cost of the concrete in place at $8.00 per cu. yd. and the cost of metal in place at 3 cts. per lb., we obtain the following costs:

With 3% reinforcement:

For cost concrete—

$11\frac{1}{2}'' \times 11\frac{1}{2}'' \times 12' = 11.04$ cu. ft. concrete.

$$\frac{11.04}{27} \times \$8.00 = \$3.27.$$

48 ft. $1\frac{1}{8}$ diam. rod, at 3.379 lbs. = 162 lbs.

162 lbs. at $0.03 = $4.86.

	$3.27
	4.86
Total cost .... =	$8.13

For 1% reinforcement:

$12\frac{1}{4}'' \times 12\frac{1}{4}'' \times 12' = 12.50$ cu. ft.

$$\frac{12.50 \times \$8.00}{27} = \$3.66$$

48 ft. $^{11}/_{16}$-in. diam. rod, at 1.262 lbs. = 61 lbs.

61 lbs. at \$0.03 = \$1.83

Total cost .... = \$5.49

The saving when 1 per cent. of reinforcement is used will be \$8.13 — \$5.49 = \$2.64, or a saving of 32.6 per cent.

The value of the ratio $\frac{E_s}{E_c}$ represented by e varies according to the quality of the concrete used. Values from 6 to 20 are used by different engineers. In general, the value of e may be taken at 10, which value probably represents the approximate value of this ratio for ordinary concrete used in compression. However, if the true value of e varies slightly from the value 10 here chosen, it will not affect materially the working stress of the concrete or the size of the column. For assuming values of e at 8, 10, 15 and 20 and substituting in Formula (6) we obtain for 1 per cent. of reinforcement:

$$\text{For } e = 8 \quad \frac{P}{f_c} = 1.08\,A$$

$$\text{for } e = 10 \quad \frac{P}{f_c} = 1.10\,A$$

$$\text{for } e = 15 \quad \frac{P}{f_c} = 1.15\,A$$

$$\text{for } e = 20 \quad \frac{P}{f_c} = 1.20\,A$$

Taking e = 10 as the basic value for e = 8, or a value of 20 per cent. less than 10, we have 1.10 — 1.08 = .02 decrease, or

$\frac{.02}{1.10}$ = 1.8 per cent. change in area.

In a like manner, for e = 15, or an increase in value of 50 per cent. over our basic value, we have an increase in section of 4.54

per cent., and for e = 20 an increase in section of 9.1 per cent. For a higher percentage of metal than 1 per cent. the change in section will be somewhat greater than that here shown, the largest increase resulting when the highest percentages of metal are used. This makes a close determination of the value of e desirable when high percentages of metal are to be used.

The method of reinforcing columns with longitudinal rods has not proved entirely satisfactory for many reasons. It is often impossible to keep the upright rods in proper position during the construction of the column. Again, rods thus embedded in concrete are not in a position to develop the full strength of the steel, as before the shortening has become great enough to develop the full strength of the concrete the metal has reached its elastic limit. The low stresses in the steel and comparatively low working stresses in the concrete not only do not give an economic section, but when heavy loads are to be carried necessitates so large a column as to be very objectionable.

While it has been admitted up to the present time that stone mortar and concrete, when subjected to direct compression, always fail by shearing along planes inclined to the direction of stress, recent experiments made by Messrs. Foeppel, in Germany, and Mesnager, in France, seem to prove that this method of failure is due to the friction between the planes of the test specimens and the plates transmitting the pressure. The friction between these surfaces was greatly reduced by greasing the surfaces of the testing machine, and it was found that failure took place not along planes inclined to the axis of pressure, but along planes parallel to the direction of pressure. If these experiments are taken as conclusive, it is evident that very little strength is added to the concrete by the use of longitudinal reinforcement. It is evident that rods in this position cannot prevent the separation of the molecules either vertically or obliquely. The total strength will then be that of the concrete, plus that of the steel.

Shrinkage Stresses.—The tendency of concrete to shrink when setting in air causes high internal stresses, tension in the concrete and compression in the steel. Considère describes experiments made in 1902 at the School of Bridges and Roads at Paris which show to some extent the stresses resulting from the shrinkage of concrete in setting.

A bar of 1:3:6 concrete 6 ft. 6 ins. long and 4 ins. square reinforced with 4 rods ¼ in. in diameter, placed near each corner, showed sufficient shrinkage of the concrete at the end of 3 months to cause a compressive stress in the steel of 6,540 lbs. per sq. in. Similar specimens 8 × 16 ins. in section by 13 ft. long, reinforced with 4 rods ⅞ in. in diameter, about $1\frac{1}{8}$ ins. from the surface, showed sufficient shrinkage to produce compressive stresses in the steel varying from 10,800 to 14,220 lbs. per sq. in. Thus we see that in compression members the stresses induced in the steel by the shrinkage strains become of so great importance that they must not be neglected in computing the strength of the member. Ordinary concrete can stand, without crushing, a reduction in length of from .0007 to .001 of its length. Such a deformation in the concrete will cause stresses to be developed in the metal which will vary from 20,000 to 30,000 lbs. per sq. in. for a coefficient of elasticity of 30,000,000 lbs. for the steel.

Adding this stress to the previous stress of say for 6,000 to 14,000 lbs. gives a total stress of from 26,000 to 44,000 lbs. per sq. in. on the metal. These stresses approximate or exceed the elastic limit of mild steel. Therefore, when the concrete is strained at or very near its ultimate strength, the steel will already have been strained beyond safety, and failure will not be delayed by the steel, but will take place suddenly and without warning. When horizontal ties are employed to bind together the vertical reinforcements at intervals about the thickness of the columns, failure takes place by the buckling outward of the rods between the ties, accompanied by local disintegration of the concrete. Hence the total strength of the column will closely approximate the strength of the concrete, plus that of the longitudinal rods when stressed up to their elastic limit. After the elastic limit of the metal is passed, the value of its coefficient of elasticity is greately reduced and its power of resistance becomes correspondingly less. The portion of the load which each will carry, as explained above, will depend upon their respective sectional areas and the moduli of elasticity of the concrete and the steel.

When longitudinal reinforcement is used the bars are immediately available for sustaining a certain amount of load as soon as they are built into the column, and before the concrete itself has hardened sufficiently to have acquired much strength. The bars are also available for spanning over places of local weakness. The adhesive bond between the concrete and the metal permits a transmission of stresses from the concrete to the steel, inducing higher stresses in the latter at points where local weakness exists in the concrete. These stresses are again transferred back to the concrete at a lower point, where it again attains its normal strength through the medium of the bond.

It does not seem advisable to depend entirely upon the bond to transmit the stresses from the steel to the concrete at the bottom of the column. A bearing plate or suitable shoe should therefore be placed under the reinforcement at the foot of the column.

Lean mixtures have much less compressive strength and are more compressible than rich mixtures. It follows that the longitudinal reinforcement will be subjected to higher stresses at all stages of loading in a column made with a lean concrete mixture than when a rich mixture is used. We may therefore conclude that steel with a high elastic limit is especially desirable for reinforcing columns when lean mixtures are used.

Under working conditions, however, neither the concrete nor the steel is highly strained and the metal often serves to transmit the strain through zones of comparatively low strength which are sometimes present in concrete. This function, which is seldom taken into account when determining the theoretical strength of concrete columns, is one nevertheless which should not be lost sight of in proportioning the columns and would lead to condemning the use of unreinforced or very lightly reinforced concrete columns so much in favor for certain classes of work at the present time. This point should be emphasized when higher working stresses than 350 or 400 lbs. per sq. in. are used.

On account of the uncertain action of columns with straight longitudinal reinforcement, it is desirable to use comparatively low working stresses. Factors of safety ranging from 6 to 10 are not too high for this kind of member.

Hooped Concrete.—The resisting power of concrete may be augmented by reinforcing it against lateral yielding either by shearing in a vertical or diagonal direction or by preventing the concrete from spreading laterally as shortening takes place under heavy loads. It has been found that when a block of concrete is subjected to heavy pressure the cohesion between the molecules is lessened as the block decreases in height and increases in size in a direction perpendicular to the line of pressure. This tendency of the molecules to flow horizontally is resisted by the cohesion and the friction between the molecules. If the lateral expansion is prevented by surrounding the concrete with a tube of metal or by confining it with spirals or hoops, its resistance to compression will be greatly increased. The maximum degree of efficiency will be reached when the hooping is continuous and of sufficient strength and rigidity to retain the component parts of the concrete within certain definite limits. As has been stated, the leaner mixtures of concrete are most compressible. Hence, only rich mixtures are suitable for use in hooped columns, as a minimum shortening in the column length is desired to bring the hooping into action as the loads are applied.

Experiments made at Columbia University by Prof. Ira H. Woolson show the manner in which concrete confined in tubes

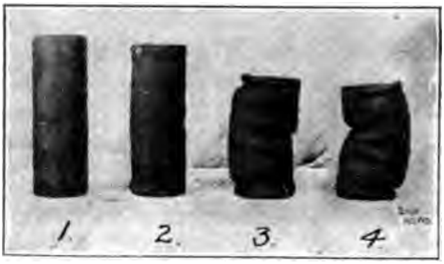


Fig. 208.—View Showing Flow of Concrete Under Heavy Pressure.

flows in a lateral direction when subjected to excessive pressure. A series of short columns, 4 ins. in diameter and 12 ins. long, were constructed by filling steel tubes of that size with finely crushed stone concrete and allowing the same to set. The concrete in the column tested was 17 days old at the time of test and appeared to be very hard. The metal of the tubes was of different thicknesses, varying from $\frac{1}{8}$ in. to $\frac{1}{4}$ in. The heavier metal columns carried a load of 150,000 lbs. (about 12,000 lbs. per sq. in.) without injury except a slight shortening of less than $\frac{1}{4}$ in. The columns with the light weight tubes began to show a marked deformation under a load of 120,000 lbs. (9,500 lbs. per sq. in.); this gradually increased until a load of 150,000 lbs. was applied, when the tests were discontinued. The photograph, Fig. 208, shows what happened in different cases.

No. 1 was unaffected by the full load, No. 2 had sustained a load of 115,000 lbs. (9,000 lbs. per sq. in.). A bulging ring at the top and bottom shows that failure had just begun. Nos. 3 and 4 show excessive deformation due to increase of load. They all compressed $3\frac{1}{4}$ ins. and $3\frac{1}{2}$ ins., respectively. The diameter had correspondingly increased to about 5 ins.

It was supposed that this excessive distortion had completely disintegrated the concrete and left it a powdered mass, but when the tube was sawed apart and removed, the concrete was found to have taken the exact shape of the distorted tube and was solid and perfect in every way possible. This experiment shows conclusively that the concrete had actually flowed under the pressure like any plastic material. The concrete was apparently perfectly dry and no signs of moisture could be observed.

A similar experiment is discussed by Considère. A hooped prism of concrete, a 1 : 3.2 cement gravel mixture, was subjected to a pressure of 7,940 lbs. per sq. in. of the original section. The prism became very much bent, taking the shape of the letter S. The maximum deflection was 0.4 ins. in a length of 13 ins. The curvature was the sharpest at the middle of the specimen, the least radius of curvature being about 2 ft. The stretched fibers showed no transverse cracks, and therefore did not suffer greatly from the extension. The computed shortening of the compression fibers amounted to 17 per cent.

The hooping and longitudinal reinforcing were removed and the concrete core, having a length of 4 ft. 3 ins., could not only be handled without breaking, but when placed on two blocks 3 ft. $7\frac{1}{2}$ ins. apart, it required a load of 55 lbs. to break it by bending. One half of the same core not so much bent was put on two supports $20\frac{1}{2}$ ins. apart, and required a load of 428 lbs. to break it. This indicates a tensile resistance at the extreme fiber of 205 lbs. per sq. in.

In another experiment a metal tube $7\frac{1}{2}$ ins. in diameter was filled with Portland cement and bent so that the radius of curvature of its neutral axis was 21.6 ins. The metal shell was then removed and a piece of cement cut out. The cement which had been subjected to this great deformation did not break and only showed a few cracks on the compression side.

Another test described by Considère was as follows: A specimen mixed with the proportions of 1 part of cement to 4.3 parts of sand and gravel withstood a pressure of 10,270 lbs. per sq. in.

with a maximum longitudinal shortening of 2.8 per cent. and an average shortening of 2.4 per cent. After the removal of the reinforcement the concrete core sustained a pressure of 924 lbs. per sq. in. Another specimen withstood a pressure of 6,970 lbs. per sq. in., with a shortening of 0.6 per cent., and after the removal of the reinforcement the plain concrete withstood a pressure of 1,420 lbs. per sq. in.

Table LXVIII. gives a summary of the results obtained by Considère in 1901 from tests on specimens of cement mortar 1.6 ins. in diameter, reinforced with a hooping of fine iron wire and without longitudinal reinforcement. The mortar was mixed in the proportions of 675 lbs. of cement per cu. yd. of sand (a 1:4 mixture by volume, assuming 1 cu. ft. of cement weighs 100 lbs.), with the exception of one prism in which the proportion of cement was 730 lbs. per cu. yd. (a 1:3:7 mixture by volume). The iron wire was drawn cold and had no definite elastic limit, but the stress of 78,200 lbs. per sq. in. corresponded with what was virtually the yield point.

TABLE LXVIII.

PROPERTIES OF HOOPED CEMENT MORTAR.

Specimen No.	1	2	3	4	5
Cement per cu. yd. of sand, lbs.	675	675	675	675	730
Proportions by volume	1:4	1:4	1:4	1:4	1:3.7
Age of mortar, days	8	14	22	23	100
Percentage of reinforcement in cross section	2	3	4	2	3.4
Crushing strength of total section, lbs. per sq. in.	4,870	6,540	7,360	4,930	10,500
Crushing strength of mortar (not reinforced) lbs. per sq. in.	569	711	853	853	2,420
Increased strength due to hooping	4,301	5,829	6,507	4,077	8,080
Calculated resistance of iron as longitudinal reinforcement	1,564	2,346	3,128	1,564	2,658
Value of hooping in terms of longitudinal reinforcement	2.7	2.5	2.1	2.6	3.0

The above experiments illustrate to some extent the remarkable ductility of hooped concrete, and the enormous pressures which it will sustain without apparently losing much of its original strength.

Considere's Experiments on Hooped Concrete.—M. Considère made a large number of experiments to determine the coefficient of elasticity of hooped concrete. Table LXIX. gives details of a few of these experiments. The test specimens were octagonal in section, having a diameter of 6 ins. and a length of 4 ft. 3 ins. The prisms were reinforced with helicoidal spirals and longitudinal rods. The amount of reinforcement and proportion of cement, both by weight and by volume, assuming 1 cu. ft. of cement weighs 100 lbs., are given in Table LXIX. The amount of shortening under given loads is shown in Table LXX.

TABLE LXIX.

MATERIAL OF TEST PIECES.

Test No.	Weight of cement to 1 cu. yd. gravel.	Proportion by volume.	Spirals. Diameter, inches.	Spirals. Spacing, inches.	Longitudinal Rods. Number.	Longitudinal Rods. Diameter, inches.
1	840	1 : 3.2	¼	0.79	8	0.3125
2	840	1 : 3.2	¼	0.79	8	0.3125
3	840	1 : 3.2	¼	0.79	20	0.276

Similar results were to be expected in the various prisms. This, however, was not found to be the case. For specimen No. 1 with the pressures below 2,845 lbs. per sq. in. gave a coefficient of elasticity of 7,110,000, while No. 2, which was of identical composition, gave a coefficient of elasticity of 2,845,000 lbs. per sq. in. This difference was due to the quantity of water used for mixing the concrete. The correct amount of water was used for No. 1, while an excessive amount was used for No. 2, giving a soft concrete, which did not acquire the compactness essential for a high coefficient of elasticity. Thus we see the coefficient of elasticity may vary within wide limits, according to the amount of water used in mixing.

Upon plotting the deflection curves it will be found that after a certain pressure has been exceeded a decided change in the inclination of the curve takes place. The point of this change may be taken as the elastic limit. Considère found that the elastic limit and the resistance to crushing are almost independent of the amount of water used in mixing, but vary according to the amount of cement used. The coefficient of elasticity was not, however, found to be affected materially by the amount of cement used.

During loading and unloading the deformations show a permanent set, which increases if the same load was repeated, but in a less and less degree, and rapidly approaches its final limit. A reduction in the temporary deformation was observed during the loadings and unloadings following the first, which appreciably increases the coefficient of elasticity. It was also observed that the deformation curves for the repeated loadings turn their concave

TABLE LXX.

EFFECT OF REPEATED LOADING AND UNLOADING.

		First Loading and Unloading.											
Pressure, lbs. per sq. in.		1,053	1,850	2,375	3,330	3,825	4,490	3,825	3,330	2,375	1,850	1,053	0
	Test No.												
Shortening in inches	1	0.0047	0.0095	0.0130	0.0158	0.0260	0.0480	0.0480	0.0378	.....	.....	.....	0.0146
	2	0.0173	0.0276	0.0350	0.0437	0.0567	0.0658	0.0658	0.0615	.....	0.0496	0.0480	0.0248
	3	0.0118	0.0205	0.0264	0.0350	0.0437	0.0445	0.0445	0.4000	0.0354	0.0335	.....	0.0102
		Second Loading.											
Pressure, lbs. per sq. in.		1,850	3,300	4,490	4,890	5,150	5,420	5,600	6,210	6,740	....	....	....
	Test No.												
Shortening in inches	1	....	.0449	.0540	.0645	.0662	.0685	.0827	.1356	.1940	Flexure at 6,958 lbs. per sq. in.		
	2	.0540	.0642	.0745	.0772	.0934	.0945	....	....	....	Flexure at 6,335 lbs. per sq. in.		
	3	.0335	.0406	.0540	.0582	.0630	.0765	.0875	....	....	Flexure at 7,940 lbs. per sq. in.		

side to the axis of pressure, while it is convex toward that axis in the curve of deformations due to first loading.

It follows that the coefficient of elasticity, which is graphically represented by the inclination of the tangent to the curve of deformation, increases with the pressure with a repetition of the loading, instead of decreasing with an increase of pressure, as under the first application of the loading.

It is evident that flexure is to be feared in a column under high pressures, and it is therefore unfortunate that the coefficient of elasticity which is directly proportional to the column resistance decreases with the increase of pressure, as is the case under the first application of the load. On the other hand, it is especially fortunate that hooped concrete which has been subjected to a first load has a coefficient of elasticity which increases as the pressure increases, providing the pressure does not exceed that of the first load. To illustrate this point a specially prepared prism was tested. This prism was octagonal in section of 4.3 ins. diameter.

TABLE LXXI.

EFFECT OF REPEATED LOADINGS.

First loading and unloading.		Second loading and unloading.		Third loading and unloading.		Fourth loading and unloading.	
Load, lbs. per sq. in.	Shortening, inches.	Load, lbs. per sq. in.	Shortening, inches.	Load, lbs. per sq. in.	Shortening, inches.	Load, lbs. per sq. in.	Shortenin inches.
128	....	1,180	.0504	1,620	.2790	1,620	.553
441	.0047	1,620	.0544	3,170	.3105	4,720	.614
810	.0142	1,990	.0615	4,720	.3400	6,340	.638
1,180	.0205	2,360	.0693	5,530	.3580	7,100	.658
1,620	.0299	3,170	.1088	6,340	.3980	7,525	.669
1,990	.0457	3,990	.1560	7,525	.6590	7,910	.684
2,360	.0630	4,720	.2240	7,910	.6600	8,710	.768
1,620	.0606	5,530	.3600	7,525	.6575	10,290	...
1,180	.0599	5,160	.3590	7,100	.6580	9,280	.950
810	.0528	4,720	.3386	6,340	.6540	7,910	.950
441	.0394	3,170	.3245	4,720	.6410	6,340	.934
128	.0299	1,620	.2980	1,620	.5830	128	.795
....	....	128	.2410	128	.5480	....	...

The concrete was a 1 cement, 1 sand and 3.2 gravel mixture. The gravel varied from 0.2 to 1 in. in size, and the sand all passed a screen having 0.2 holes. The helicoidal spirals were of iron wire 0.17 in. in diameter, the adjacent coils were approximately 0.82 in. centers and arranged 3¾ ins. in diameter. Eight longitudinal wires were also used, of the same size and material as the hooping metal. The length of this prism, 51.18 ins. Table LXXI. gives the loading and resulting deformation.

After the high pressure of 10,290 lbs. per sq. in. had been applied and removed it was found that the coefficient of elasticity was as high as after the application of the lightest pressure.

Considère draws the following conclusions in regard to the coefficient of elasticity: "The application of a first pressure on a hooped prism, no matter how high the pressure may be, as long as it is below the breaking load, has the effect of raising its elastic limit up to that pressure. The coefficient of elasticity which is subsequently developed by the hooped concrete under all the variations of the pressures between the lowest and the previously applied load is higher than the highest coefficient of elasticity which the prism had before the test load, and which held true for a low pressure only. The increase in the coefficient of elasticity of the concrete after the test load, as compared to the coefficient before, is so much greater the less the proportion of cement and the lower the quality of the concrete."

Elastic Behavior of Hooped Concrete.—To determine definitely the effect of hooping, it is necessary to test identical prisms with and without hoops. As it is not possible to make identical prisms with and without hoops, one of two methods must be followed in making a test. The test specimens must be prepared as nearly alike as possible and corrected for their differences in the initial values, or a prism may be tested with its spirals and then tested again after the spirals have been removed. By means of such tests Considère determined that the increase of the coefficient of elasticity due to hooping was practically equal to 90 per cent. of the coefficient of elasticity of longitudinal rods having the same weight of metal.

It is generally the case that concrete setting in the air shrinks or contracts. On account of this shrinkage or contraction it is evident that the concrete will not bear effectively against the wire hooping until the load has been applied and the swelling has begun, thereby bringing the concrete and the spirals into close contact. It has been found that the hooping does not come into operation until the pressure per square inch reaches a limit of from 1,200 to 1,500 lbs. per sq. in., depending upon the richness of the concrete. The above condition exists before the application of the first load. After the first load has been applied, however, the concrete remains in close contact with the spirals and the hoops have their normal effect upon the subsequent application of loads.

Taking into consideration these facts, it would appear that there is considerable difference between the action of hoops and longitudinal reinforcing rods. As has been stated, longitudinal rods are compressed by the shrinking of the concrete, are brought immediately into action upon the application of the loads and rapidly reach the elastic limit of the metal. For steel this limit is reached when the longitudinal shortening becomes about .06 per cent. of the length. The hoops compressed by the shrinkage must, on the contrary, first return to a state of molecular equilibrium before they take tension, and this tension only becomes important when their longitudinal deformation, i. e., lengthening, has reached values above 0.06 per cent. of its length. The hoops have only begun to be seriously stressed under a first application of the load in prisms hardened in air when the longitudinal rods have already passed the elastic limit, are almost at their ultimate strength and cannot offer any further resistance.

It was observed that the hooping did not produce its normal effect on the elastic behavior of the concrete until the load approached 220 lbs. per sq. in. It was also observed that after the first application of the load a permanent swelling existed in the concrete, which brought the hooping into action immediately upon the application of subsequent loads.

The action of the hooping extends through a wider range than that of longitudinal rods, the elongation of the hoops or spirals resulting from the swelling of the concrete, which is small, varying probably from 0.3 to 0.4 of the longitudinal shortening. The stress in the metal being proportional to its deformation within its elastic limit, it is evident that after having begun to deform the deformation is much slower for the hoops than the longitudinal rods. As has been explained, this resisting action of the hooping greatly augments the resistance of the column.

When concrete is gradually hardened under water it expands, bringing tension upon the spirals and upon the longitudinal rods. The spiral hooping will be brought into play at once when the load is applied, while the tension in the longitudinal rods must first be overcome before the compression is begun.

The elastic limit of hooped concrete members under a first load evidently depends upon the elastic limit of concrete, which is reached before that of the metal. It may be admitted that the shortening of the concrete increases greatly under high pressure and that practically the elastic limit is reached when the shortening becomes equal to from 0.08 to 0.13 per cent. of the length of the member under load, depending upon the character of the concrete.

Summarizing, we have the following rules in regard to the coefficient of elasticity and elastic limit of hooped concrete:

(1) For the first load, the coefficient of elasticity of a hooped member is equal to the sum of the coefficients of the concrete, of the longitudinal rods, and of the imaginary longitudinals, whose volume shall be assumed as 90 per cent. of the hoops or spirals.

(2) For pressure less than a previous test load, the coefficient of elasticity of a hooped member is equal to the sum of the coefficients of the concrete, as increased by the test load, the existing longitudinal rods and of imaginary longitudinals whose volume shall be assumed as double that of the hooping or spirals.

(3) The elastic limit of a hooped member, for a first load, is equal to the natural elastic limit of the concrete, increased by the resistance of the reinforcing as found for a shortening of 0.0008 to 0.0013 and computed on the basis indicated above for the coefficient of elasticity under a first load. Every load has the effect of making the final elastic limit practically equal to the pressure due to this load.

Experiments were made by Considère to determine the increase in strength due to hooping, by confining sand in a tube. It can be easily shown that the resistance given to the sand by the steel is 2.4 times as much as would be offered by longitudinal reinforcing rods of the same weight as the shell when the tensile stress in the former is equal to the compressive stress in the latter. This gives the ratio of the coefficients of the two types as 2.4 to 1.1. This 2.4 is also the ratio of the crushing resistance of the two types of reinforcement for equal weights of reinforcing metal. This is true because crushing takes placed in hooped members reinforced longitudinally when the elastic limit of the metal has been reached, which is the same for tension as for compression.

Compressive Resistance.—The compressive resistance of a hooped member exceeds the sum of the following three elements:

(1) The compressive resistance of the concrete without reinforcement.

(2) The compressive resistance of the longitudinal rods stressed to their elastic limit.

(3) The compressive resistance which would have been produced by the imaginary longitudinals at the elastic limit of the hooping metal, the volume of the imaginary longitudinals being taken as 2.4 times that of the hooping metal.

An indirect method of determining the strength of a column reinforced both longitudinally and spirally, is as follows. Let A_s^h equal the sectional area of an imaginary longitudinal reinforcement equivalent to the hooping. Then the strength P of the column will be

$$P = F_c\ (A_c + e\,A_s + 2.4\,c\,A_s^h).$$

Let F_s^c = elastic limit of metal in compression and F^s_s that of hooping in tension and F_c the ultimate strength of concrete in compression. Then

$$P = F_c\,A_c + F_s^c\,A_s + 2.4\ F_s^s\,A_s^h.$$

A more direct method of determining the strength of hooped concrete is given on page 402. In the use of the formula there given the strength of the longitudinal rods is neglected, it being assumed that they care for secondary stresses only.

Spacing of Hoops.—Experiments were made by Considère to determine the most favorable spacing of the hoops or spirals. It was found that when the spacing of adjacent spirals did not exceed $^1/_5$ the diameter of the coils, resistances were obtained almost independent of the spacing. These facts, with others which have been observed, lead to the adoption of a spacing of spirals of from $^1/_7$ to $^1/_{10}$ of their diameter when longitudinal reinforcing rods are also used. Numerous experiments have demonstrated that the above ratio holds true almost independently of the absolute value of the dimensions.

Additional experiments were made by Considère, assisted by Messrs. Mesnager and Mercier, to determine the resistance of concrete subjected to radial pressure. It was found that the compressive resistance of a prism of mortar or concrete per unit of area equals $1.5\ f_c + 4.8\ p$ when f_c is the natural unit compressive resistance of the concrete, and p is the unit pressure exerted by the hooping on the whole of its lateral surface.

The coefficient 4.8 represents the compressive resistance of a prism of the same dimensions consisting of particles of the same concrete without any cohesion whatever. This resistance is thus due to friction only.

The term 1.5 f_c, which is of greater importance the higher the specific resistance of the material, is due to the cohesion between the particles of the concrete. This cohesion increases as the particles are forced closer together by the increasing pressure. The increase in the resistance due to this cause is produced gradually only, and is proportional to the increase of the lateral pressure. It reaches its greatest value when the pressure amounts to from 60 to 70 lbs. per sq. in. The coefficient of f_c equals unity when the pressure p equals zero, and only becomes equal to 1.5 when p equals from 60 to 70 lbs., and is maintained for all higher values of p. For hooped columns a coefficient value of 1.5 should always be used, as the lateral pressure due to hooping under practical working conditions is never less than 60 lbs. per sq. in.

Working Formula for Hooped Concrete.—A practical working formula may be developed from the empirical expression for pressure as determined by Considère and given above, together with well-known principles of hydrostatics.

Let R = pressure per square inch on prism,
d = diameter of prism and hooping,
z = distance between adjacent coils of hooping,
p = external radial unit pressure,
= the uniform unit pressure exerted by the hooping,
f_c = natural unit compression stress of concrete,
f_s = unit tensile stress of hooping,
A = sectional area of prism,
A_s = sectional area of hooping metal,
R_c = unit compressive resistance of concrete due to hooping.
$R_c = 1.5\ f_c + 4.8\ p.$

From the principles of hydrostatics we know that the pressure upon a liquid in a cylinder is exerted equally in every direction and tends to tear the shell of the cylinder apart longitudinally. The swelling of concrete under pressure, neglecting the friction between the particles or molecules of the concrete may be considered as acting in the same manner as a liquid retained by a cylinder shell, and the tendency to rupture in a longitudinal direction will be resisted by the hooping.

Now, the interior pressure due to the swelling which we consider as hydraulic pressure, is resisted by an equivalent external pressure p per square inch up to the point of rupture of the hooping. The force which tends to cause longitudinal rupture is R d z. This follows from the principle of hydrostatics that the pressure of a liquid in any direction is equal to the pressure on a plane normal to that direction. Now the tensile stress of the hooping which resists the internal stress is $2 A_s f_s$. But these two quantities are equal, and we have $2 A_s f_s = R d z$. But that equilibrium shall obtain, the internal pressure R must be resisted by an equivalent external pressure $R = p$.

Then

$$p = \frac{2 A_s f_s}{d z}$$

Substituting this value of p in the equation $R_c = 1.5 f_c + 4.8 p$ we obtain the equation

$$R_c = 1.5 f_c + \frac{9.6 A_s f_s}{z d} \quad \ldots\ldots\ldots\ldots\ldots\ldots (a).$$

The total strength p of the prism will then be

$$P = A R_c = A \left(1.5 f_c + \frac{9.6 A_s f_s}{z d}\right)$$

but $A = \frac{1}{4} \pi d^2$, and

$$P = 1.178 f_c d^2 + \frac{7.54 A_s f_s d}{z} \quad \ldots\ldots\ldots\ldots\ldots\ldots (b).$$

In most cases formula (a) will be used for given values of d z, f_c, A_s and f_s. When the pressure R_c is obtained the total strength of column will then be found by multiplying this pressure by the sectional area of the column. This formula will be found to give rather higher values than have thus far been used in this country. The pressures used for a number of structures are given in another chapter. It will be well when heavy loads are carried and it is desired to use strains approximating those given by this formula to not allow the length to exceed 10 or 12 diameters.

The above formula gives high average unit stress on the sectional area of the concrete, and it would appear rational to introduce a constant in the formula to reduce the stress somewhat. This is necessary when we consider that the stiffness of hooped concrete does not increase as the strength is increased by the hooping. Let Q represent such a constant the formula (a) be- and (b) becomes

$$R_c = Q\left(1.5\ f_c + \frac{9.6\ A_s\ f_s}{z\ d}\right)\ldots\ldots\ldots\ldots\ldots\ldots(a)$$

and B becomes

$$P = Q\left(1.178\ f_c\ d^2 + \frac{7.54\ A_s\ f_s\ d}{z}\right)\ldots\ldots\ldots\ldots\ldots(b).$$

For 1:2:4 concrete Q may be taken as 0.6.

Example.—Determine the strength of a column 10 ins. in diameter, allowing a working strength of 400 lbs. per sq. in. on concrete, and 15,000 lbs. on steel, using ¼-in. round rods for the spirals. Let us assume that the spacing of hoops will be $^1/_7$ of the diameter, then $z = {}^{10}/_7$ ins. Substituting in formula (a) we have

$$R_c = Q\left(1.5 \times 400 + \frac{9.6 \times .0625 \times 15{,}000}{\frac{10}{7} \times 10}\right)$$

$= Q\ (600 + 630) = 1{,}230$ lbs. per sq. inch; or, taking Q at 0.6, $R_c = 738$ lbs.

$$A = 78.54 \text{ sq. ins.}$$
$$P = 78.54 \times 738 = 57{,}960 \text{ lbs.}$$

Concrete Columns in the Light of Recent Tests at Watertown Arsenal.—The tests of concrete columns being made at the Watertown Arsenal should, when completed, furnish sufficient data to determine the proper way in which the concrete should be used for columns. The tests thus far made, as reported by James E. Howard in a paper read at the June, 1906, meeting of the American Society of Testing Materials, are of great interest and deserve consideration in this place.

The columns tested were 8 ft. in height and from 10 to 12 ins. in diameter, and were composed of various mixtures, all being what is known as wet mixtures and all hardened in the air.

During testing the columns were loaded with increments of 50 lbs. per sq. in., measuring the amount of compression under each increment, returning to the initial load and observing the sets. Micrometer observations were made on a gauge length of 50 ins. equally distant from the ends of the columns. Full details of the tests are published in the reports of tests of metals, a Congressional document issued by the Ordnance Dept., U. S. Army. Details of these tests cannot be given in full, but such information as is necessary to illustrate the work being done will be shown.

Figure 209 represents the compressive strength of a number of mortar columns, plain and reinforced, with longitudinal bars of ¾-in. twisted steel. The progressive loss in strength of the plain

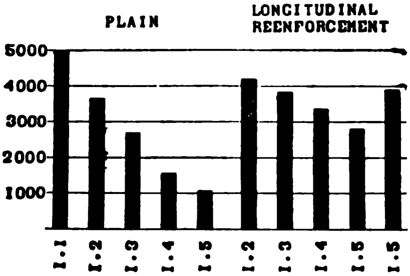


Fig. 209.—Column Tests, Watertown Arsenal.

columns as the mixture became leaner should be noted. The ultimate strength of the 1:1 column was not reached as it exceeded the capacity of the testing machine. This mixture was not reinforced. Each of the others were reinforced, four with 8 bars each and one with 13. The percentage of reinforcements was about 2.86 and 4.63. The darker shaded lower ends of the

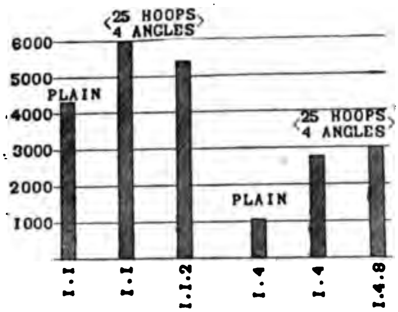


Fig. 210.—Column Tests, Watertown Arsenal.

figures represent the relative amount of the reinforcement. The steel reinforcing bars extend from end to end of the column and had a full bearing at the ends. They had no other lateral support than that afforded by the mortar in which they were embedded. A slight gain due to reinforcement of the richer mixture should be noted.

Figure 210 shows the strength of a rich and a lean mortar, each of which was reinforced with hoops and longitudinal angle bars; also corresponding concretes reinforced. The hoops measured 1.5 × 0.12 ins. in cross section, with lapped and riveted joints. The plain 1:1 mortar displayed a compressive strength of 4,320 lbs. per sq. in., which in the hooped columns run to 5,980 lbs.; the addition of two parts trap rock to this mortar resulted in a strength of 5,433 lbs.

The weaker 1:4 mortar was raised by hooping and angles from 1,050 lbs. to 2,766 lbs. per sq. in., which in a corresponding concrete reached an ultimate strength of 3,002 lbs. The free span between the hoops, 2½ ins., permitted this lean mortar to flake off, while the larger pieces of the stone in the concrete were held in place. The great increase in strength due to hooping is noticeable in the figure.

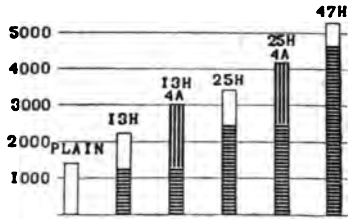


Fig. 211.—Column Tests, Watertown Arsenal.

Figure 211 shows the results of tests on 1:2:4 mixtures with various amounts of reinforcement with hooping and longitudinal angle bars. The compressive strength of this group are as follows:

	Lbs. per sq. in.
Plain column	1,413
13 hoops	2,232
13 hoops, 4 angle bars	3,029
25 hoops	3,428
25 hoops, 4 angle bars	4,189
47 hoops	5,289

Thus both the hoops and the angles contribute toward increasing the ultimate strength of the column. Any desired strength may be attained by means of lateral reinforcement if sufficient metal is used, but it is obvious that a certain amount of longitudinal compression of the concrete will be necessary before the lateral reinforcement becomes effective, which in the case of lean mixtures involves considerable deformation in the column.

Figure 212 represents several columns which are strong by reason of their composition or on account of their reinforcement. These columns in composition, reinforcement, and strength were as follows:

		Lbs. per sq. in.
1:1 mortar, plain	above	5,011
1:1:2 concrete, plain		3,900
1:2 mortar, 8¾ ins. twisted steel bars		4,200
1:5 mortar, 13¾ ins. twisted steel bars		3,905
1:2:4 concrete, 25 hoops and 4 angle bars		4,189
1:3:6 concrete, 25 hoops and 4 angle bars		3,862
1:4:8 concrete, 25 hoops and 4 angle bars		3,002

The relative rigidity of these columns, which is not suggested by a comparison with their compressive strengths, is indicated in Fig. 213. The order in which the compression curves appear is the same as shown in Fig. 209, excepting the 1 : 1 mortar and the 1 : 1 : 2 concrete, which change places, the latter appearing

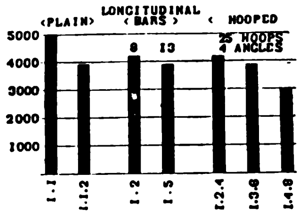


Fig. 212.—Column Tests, Watertown Arsenal.

first on the left of the group. As may be noted, the plain columns display the greatest rigidity of the several types here represented. This has been a noticeable feature in the tests as a whole.

It is even found that the plain columns are a little more rigid over the range of stresses here plotted than in the same mixtures in which longitudinal rods are used as a means of reinforcement. In so many cases has this occurred that some explanation should be sought why the presence of the steel bars, themselves so much more rigid than the concrete, should not result in increased rigidity of the column as a whole. It is not improbable that the settlement in the height of the column is so far restricted by the steel bars that minute fissures are developed during the early stages of the hardening of the concrete. Internal strains without the presence of fissures would hardly account for this behavior. The hooped columns were found to be decidedly more compressible than the others. It should be noted that lateral reinforcement, while effective in raising the ultimate strength of loads once applied, does not result in imparting rigidity to weak concrete.

These tests seem to indicate that a high ultimate strength will be reached, *first* by the use of rich mortars or concretes, *second* by means of sufficient longitudinal metal reinforcement, and *third* by means of sufficient hooping or other external lateral support. It also appears that rigidity of columns is best secured by

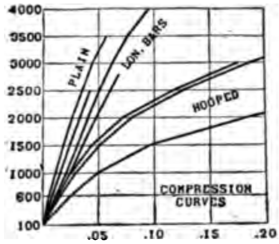


Fig. 213.—Column Tests, Watertown Arsenal.

the use of rich mixtures, although the same results may be obtained by the use of longitudinal steel bars. The use of plenty of cement seems to be the best means of securing a rigid column of high ultimate compressive strength. Hooping, while it increases the ultimate strength, does not increase the rigidity of the member. It should, however, be remembered that hooping has the great merit of increasing the elasticity of the concrete and preventing sudden and dangerous failure, thereby enabling lower factors of safety or higher working stresses to be used.

CHAPTER XXII.

FOUNDATIONS.

The most suitable foundation for use in a given locality, whether it be for a building, bridge or other structure, can not be determined by text-book rules, but is a question of engineering judgment based upon practical experience and cost. The latter item is often the controlling factor in determining the kind of foundation to be used. Concrete has been widely used for foundations, and when the economy resulting from the use of reinforced concrete is more fully understood its use will become more general.

It is not possible within the limits of this work more than to lightly touch upon the principles governing the choice of foundations to be used in any given case. The character of the underlying rock or soil determines largely the unit loads which can be safely brought upon it. It is customary to speak of a given soil as having a carrying capacity of a certain number of tons per square foot.

Bearing Power of Soils.—Firm ledge rock may in general be said to be able to safely carry any load which may be brought upon it. The greatest loads will, however, not exceed from 30 to 50 tons per sq. ft. From this maximum load for ledge rock the carrying capacity decreases as we pass from firm ledge rock to the softer varieties of rock, hardpan, gravel, sand and clay, and finally to semi-fluid materials like mud, silt and quicksand, which have little or no bearing capacity unless special treatment is resorted to.

Different classes of soil require different treatment, and when two or more kinds of material are met with in the same foundation the problem becomes indeed complex, and trained judgment and experience are needed to determine what is best to use under the given conditions. A careful examination should be made of the soil to determine its nature, its compactness, the amount of water which it contains, etc. A careful survey of the surrounding conditions often helps to determine what should be adopted for a given foundation.

Where the strata are of considerable thickness and uniform in character over extended areas, the bearing powers decrease as follows: Hard ledge rock, soft varieties of rock, hardpan, cemented gravel and sand, indurated clay, dry clay and sand, wet clay, moderately wet sand, loam, mud, silt and quicksand. Probably the most generally accepted values for safe bearing power of soils are those recommended by Prof. Ira O. Baker, which are as follows: Minimum load on rock having a hardness equal to the best ashlar masonry, 25 tons per sq. ft,; for rock equal to the best brick masonry, 15 tons; for rock equal to poor brick masonry, 5 tons; for dry clay, 4 tons; for moderately dry clay, 2 tons; for soft clay, 1 ton; for cemented gravel and coarse sand, 8 tons; for compact and well cemented sand, 4 tons; for clean dry sand, 2 tons, and for quicksand and alluvial soils, not more than ½ a ton per sq. ft. The above values may be increased from 25 to 100 per cent., depending upon circumstances and judgment of the engineer.

The following are the regulations of the New York Building Code in regard to Bearing Capacity of Soil:

"When no test of the sustaining power of the soil is made, different soils, excluding mud at the bottom of the footings, shall be deemed to safely sustain the following loads to the superficial foot, namely: Soft clay, 1 ton per sq. ft.; ordinary clay and sand together in layers, wet and springy, 2 tons per sq. ft.; loam, clay or fine sand, firm and dry, 3 tons per sq. ft.; very firm coarse sand, stiff gravel or hard clay, 4 tons per sq. ft., or as otherwise determined by the Commissioners of Buildings having jurisdiction." When it is desired to carry greater loads than the above, it is customary to make a test of the sustaining power of the soil.

Where it is possible the loads used in a given locality should be ascertained and a safe precedent followed. The following examples will be of interest in this connection: Soil tests were made to determine the bearing power of the blue clay subsoil at the site of the capitol at Albany, N. Y. It was found to sustain 6 tons per sq. ft. A maximum load of 2 tons was used. The building is situated on a hillside, and cracks in the side walls show that some settlement has taken place under the wall on the downhill end. A load of 2½ tons was used on yellow clay which tested to 13½ tons for the foundation of the Congressional Library at Washington, D. C. A load of 3 tons was used for the Bismark

Bridge over the Missouri River for the Missouri Pacific R. R. This clay was of a hard variety, resembling rock and tested to 15 tons. A live load of 5½ tons was used on sand overlying rock in the foundation of the Brooklyn Bridge, and 3.63 tons was used on coarse gravel 12 ft. above rock for the foundation of the Roebling Suspension Bridge at Cincinnati, O. On London clay a load of 6½ tons was used for the Cannon St. Bridge, and 9 tons for the Charing Cross Bridge. Both bridges settled. In the construction of the Tower Bridge, a test cylinder settled under 6½ tons. The loading used was 4 tons. If the skin friction and buoyancy of water are deducted, the bearing capacity will be from 1 to 2 tons per sq. ft. The soil under Washington Monument is fine sand, and carries a load of 11 tons per sq. ft. This is increased by wind pressure to 14 tons per sq. ft. In the construction of the reinforced concrete building at Cincinnati known as the Pugh Power Building there were no signs of settlement under a test load of 6 tons per sq. ft. on compact gravel, and a soil composed of part gravel and part sand stood a load of 4 tons. Loads of 5 and 3 tons, respectively, were allowed for foundations and footings up to 11 ft. square.

Foundations may be built on sand in spite of the Biblical saying. Sand, when confined, is practically incompressible, and if it can be kept free from water no danger need be apprehended from building upon it. Likewise, dry clay makes a good foundation, and when possible if it is proposed to build on clay the foundation should be drained or kept free from the action of water.

General Considerations in Regard to Foundations.—In the preparation of the foundation bed the excavation should be carried below the frost line. Again, the deeper the foundation is carried the less chance there is of displacement due to adjacent excavation, and, in general, the firmer will be the soil. Whether the foundation be on rock or on some kind of earth, the foundation bed should be cut horizontal. When the ground has a slope, steps with horizontal benches should be cut to bring the bearing upon a horizontal bed.

It is desirable as far as possible to have uniformity of material in the foundation bed. When not possible, some provision should be made so that settlement, if any at all takes place, will be uniform under the whole foundation. The character of the structure will in a large measure determine the allowable pressure on the foundation bed. Thus, where quiescent loads are to be provided for, higher loads are allowable than when moving loads or impact and vibration will exist. Thus a much less bearing power should be assumed for the piers of a railroad bridge than for an ordinary highway bridge or a building.

When plain concrete is used for foundations the usual practice is to place a bed 1 ft. or more in thickness under the walls or piers and having a sufficient width to properly transmit the loads to the subsoil. If the soil is firm enough to stand, no boxing will be needed, the excavation simply being made of the required size. Under other conditions forms will be needed to hold and protect the concrete when it is put in place.

Pile Foundations.—In many situations it is neither desirable, safe or economical to use spread foundations. The materials may be such as are not able to bear the weight of the structure after spreading the footings, or the cost of securing a proper spread of footing may be excessive, or again the subsoil may at some future time be exposed to the scouring action of water, or if of a semi-fluid nature may be disturbed by adjacent excavations. Under such conditions piles are in many cases used, giving a foundation which can be rapidly and economically put in place. Wooden piles must be cut off under water, as when subjected to an atmosphere which is alternately wet and dry, they will decay. On this account they cannot be used in many situations where otherwise they would be desirable. Concrete piles, on account of their durability, may be used under such conditions, as well as under any in which wooden piles may be driven.

Piles are driven usually 2 or 3 feet centers, in clusters or rows, depending upon whether they are to support a pier or a wall. The nature of the soil, loads to be carried, and size and length of pile to some extent govern the spacing of the piles. It has been found, however, that little or no additional bearing power is secured if the spacing is much less than 2 ft. centers. Short piles are sometimes used to compact the earth, thereby so increasing its bearing capacity as to enable it to support the weight of the superstructure. Usually, however, dependence alone is placed upon the pile to carry superimposed loads. The nature of the soil, loads to be carried, etc., determine the character of the pile foundation to be used for each case.

The Bearing Power of Piles.—The bearing power of a pile may depend upon the friction of the soil through which it is driven, upon the supporting power of the substratum in which its point rests, or upon both. The frictional value will depend upon the kind and nature of the soil through which the pile is driven, and when concrete piles are used, to a certain extent upon the rougnness of the surface of the pile. When the pile is driven through soft earth into firm, compact material, it will act as a column, and when the bearing power of the sub-stratum is high, its supporting power will depend upon its strength as a column. Of course, if driven through a firm, gritty material, the frictional value will be much higher than when the material is soft or semi-fluid, and the supporting power correspondingly increased. When the supporting power depends upon friction it is probable that it increases for a time after the driving ceases. Patton states that piles driven in the alluvial soils of the swamps of the South for railroad trestles, when forced to place by the weight of the piledriver hammer alone, or at most by a few blows with a short fall, after resting a few days were so firmly supported that it was impossible to move them by repeated blows from the pile driver hammer. On the other hand, if the supporting power depends largely upon the resistance to penetration of the sub-stratum into which the pile is driven, it is probable that the safe bearing power will decrease, as most materials require less force to change their form slowly than they will sustain for a short time.

The bearing power of a pile in any given soil is no criterion of what a similar pile will carry in any other soil, and experience and experiment must be relied upon in each particular case to determine what are the safe allowable loads. Thus an extended experience in the use of piles enables the engineer to judge closely from the manner in which piles drive in a given soil the allowable loads which may be placed upon them. When experience is lacking it may be supplied by experiment. Thus, by testing one or more piles driven in a given soil by applying a direct load or pressure upon them the maximum load which they will carry may be determined, and a safe working load chosen. When the maximum load is known it may be expressed in terms of the depth driven, kind of soil and size of pile or surface in contact with the soil. The bearing power in a given soil of a pile of any size may thus be determined. An ample factor of safety should be used when the maximum bearing power is known. This may vary from $1\frac{1}{2}$ to 10, depending upon the conditions and character of load to be supported.

When the approximate bearing power of the sub-stratum into which the point of the pile is driven is known, together with the frictional resistance of the surface of the pile in contact with the soil, the relation between these resistances and the weight which the pile will carry may be expressed by a formula. Thus, let W = weight carried by pile, p = bearing power of soil under point of pile, s = surface in square feet of pile in contact with the soil, and f = a factor depending upon the frictional resistance of the material on the surface of the pile, then

$$W = p + fs.$$

If we know p and f in all cases and the load to be carried, we can determine the depth to which a pile or group of piles must be driven to carry the given load. The value of p varies from zero for silt to from 2 to 3 tons for sand, gravel or clay. When the sub-stratum is hardpan or other firm material, a higher value may obtain. The value of f may be determined by experiment. Patton recommends the following values: 100 lbs. per sq. ft. for softest semi-fluid soils, 200 lbs. for compact silt and clay, 300 to 500 lbs. for mixed earths with considerable grit, and from 400 to 600 lbs. in compact sand and sand and gravel.

Thus, if a pile is driven through compact silt and clay into a clay sub-stratum, assuming p = 3 tons = 6,000 lbs., and 22 tons is to be carried = 44,000 lbs. if pile is assumed to have an area at the point of .75 sq. ft., p = 4,000 lbs.

Assuming f = 500 lbs., we have for value of S:

$$44{,}000 = 4{,}000 + 500\ S,$$

or

$$S = \frac{44{,}000 - 4{,}000}{500} = 80 \text{ sq. ft.}$$

or, assuming an average diameter of 12 ins. the area per lineal foot equals 3.14 sq. ft. and the pile must be driven to a depth of

$$\frac{80}{3.14} = 25.5 \text{ ft.}$$

Formulas in common use for determining the bearing power of piles are in general based upon the relation existing between the supporting power of the pile, the length and size of the pile, the weight of hammer used in driving, height of fall and distance the pile was moved by the last blow, or average distance of several last blows of pile driver hammer. When this relation can be expressed by an equation, the supporting power can be found by inserting these quantities in the formula and solving it. The relation between these quantities must be determined from a consideration of the theoretical conditions involved. Numerous formulas have been evolved by different engineers which differ greatly as to results obtained. The limits of this work preclude a discussion involving the merits of different formulas. We will therefore only give a single formula, which, on account of its simplicity and the safe results obtained from its use has gained great popularity. This formula is known as the Engineering News Formula, and when used in the following form gives a factor of safety of 6:

$$P = \frac{2\,W\,h}{d + 1},$$

in which P = safe load in tons, d is the penetration in inches under the last blow, or, better still, the average of several last blows, W = weight of hammer, and h = height of fall in feet.

Thus, if W = 2 tons, h = 20 ft., d = 3 ins.,

$$P = \frac{2 \times 2 \times 20}{3 + 1} = 20 \text{ tons.}$$

It is probable that the above formula is not in general applicable to concrete piles. Reliance should therefore be placed alone on actual tests to determine the bearing power of concrete piles. A number of examples are cited on later pages of this chapter in regard to loads placed upon concrete piles. These should be referred to in this connection.

Reinforced Concrete Foundations.—Reinforced concrete is a material well adapted to the construction of foundations for buildings, bridges, wharves, docks, etc. It is used in the construction of spread foundations for high buildings, and as a capping for timber piles. In the construction of sheeting and bearing piles it possesses many valuable characteristics and undoubtedly will have an extensive use in the future. This material has also been used as a sheathing material for timber piles in teredo-infested waters and as a protection for steel piles used in pier construction.

Spread Foundations.—Spread foundations are either isolated column footings, combined footings for two or more columns, wall footings or continuous footings extending over the whole foundation area. In whichever form it is used, the purpose of the foundation footing is so to distribute the load over the soil that its carrying capacity will not be exceeded.

Among the advantages of this material for spread foundations are a reduction in the amount of excavation required, a saving in material and a reduction in the weight of the foundation itself, thereby greatly reducing the cost of the substructure for a given construction.

The simplest form of spread foundation, and the earliest used, is not, strictly speaking, a reinforced concrete construction, but a steel construction surrounded and protected by concrete. This is the grillage beam foundation. The earliest form of beam grillage consisted of steel railway rails superimposed in layers at right angles to each other and embedded in concrete. A later construction consisted of replacing the top layer of rails with steel I-beams. Sometimes I-beams are used for the whole foundation. The lengths of the successive tiers of beams decrease from the bottom to the top. Usually the column footing consists of a cast shoe resting upon the topmost layer of beams, although either stone footings or built-up steel shoes may be used.

The method of construction of grillage beam foundations is as follows: The ground is excavated to the proper depth and carefully leveled off. If the character of the ground is not such that it will remain vertical without the sides of the pit falling in, boxing of the exact size of the footing is constructed. This boxing is accurately centered and its sides carefully leveled to the proper elevation. The concrete is then put in and tamped in layers of 6 or 8 ins. and the top leveled off even with the top of the boxing. The steel beams should then be carefully bedded in 1 to 2 Portland cement mortar so they will be as nearly level as possible. The beams are set one after another and the spacers placed as the work proceeds, or, if they are not used, care is exercised to place the beams parallel and at proper intervals apart. As soon as the beams are in place the spaces between them are filled with concrete. When the beams are very close together grout may be used. This filling should be carefully done and the concrete well tamped so that the top flanges of the beams will have a firm bearing upon it. A boxing like that used for the footing is placed about the beams. The spaces at the sides and ends of the beams are filled with concrete, and a coat of mortar is plastered over the top. The next layer of beams is placed in exactly the same manner, and so on until the top layer is set. Upon this the stone or metal base plate or column footing is set in a bed of mortar. A layer of at least three inches of concrete should then be spread over the top of the beams, and the whole coated with 1 to 2 Portland cement mortar.

This form of foundation has been extensively used for more than 20 years in building construction and is sometimes called

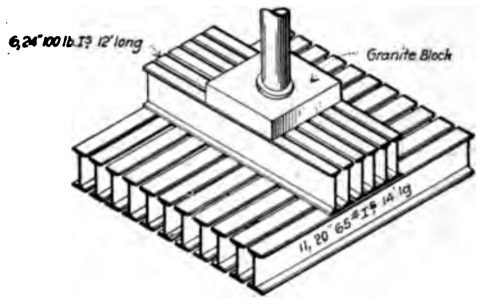


Fig. 214.—Beam Grillage Column Footing.

the Chicago Foundation. Fig. 214 shows a typical column foundation of grillage beams which was used in the construction of the Franklin Building in New York. The bottom tier of beams consists of eleven I-beams 20 ins. × 65 lbs., 14 ft. long, and the top tier of five I-beams 24 ins. × 100 lbs., 12 ft. long.

The simplest form of reinforced concrete spread foundation consists of a simple column or wall footing of concrete reinforced at the lower or tension face by a series of iron or steel rods or some form of netting, as shown by Fig. 215.

The Monier construction, when used for spread foundations, consists of the regular Monier network embedded in the tension side of the concrete. In wall foundations the carrying rods are perpendicular to the axis of the wall and should be proportioned to support the load which comes upon the foundation. The distribution rods are placed parallel to the axis of the wall. It is customary to proportion them so that the foundation wall may have a girder action thereby enabling it to span openings in the wall and give an even distribution of the load to the foundation. For column footings both sets of bars are carrying bars. Fig.

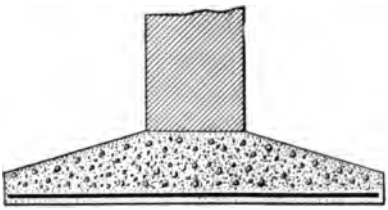


Fig. 215.—Typical Reinforced Concrete Column Footing.

216 shows a typical column footing. This particular footing was used in the Robert Gair factory building, Brooklyn, N. Y.

When heavy loads are to be carried it is customary to use two or more layers of netting, placing one above the other, with layers of concrete between the successive nettings. Expanded metal

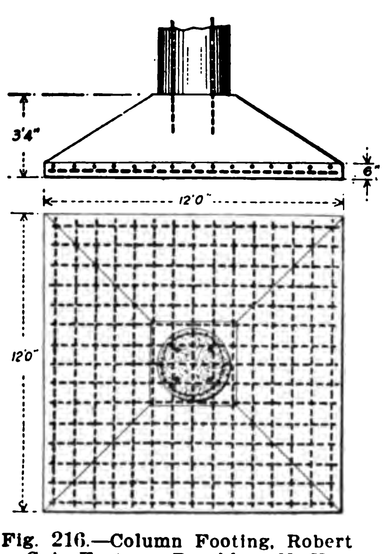


Fig. 216.—Column Footing, Robert Gair Factory, Brooklyn, N. Y.

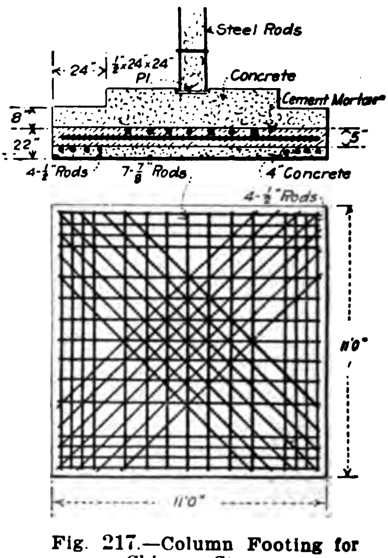


Fig. 217.—Column Footing for Chicago Store.

is used in much the same manner as the Monier netting. The arrangement and location of the bars are practically the same in all systems. The bars are placed at right angles to each other, sometimes diagonal bars are also added and located as near as possible to the lower face of the concrete. A sufficient thickness of concrete must be placed below the rods to protect them from the soil and water. When great strength is required a reinforcement is sometimes placed in the compression side of the footing. Figure 217 shows the reinforced concrete column footing used in a store building in Chicago, designed by Mr. Lee Heidenreich.

Two or more columns are sometimes placed upon one footing. Under these conditions care must be taken in determining the shape of the footing that the load may be distributed uniformly

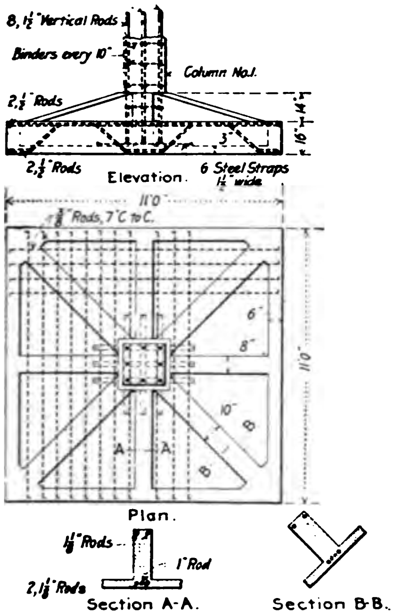


Fig. 218.—Column Footing for Atlanta, Ga., Terminal Railway Station.

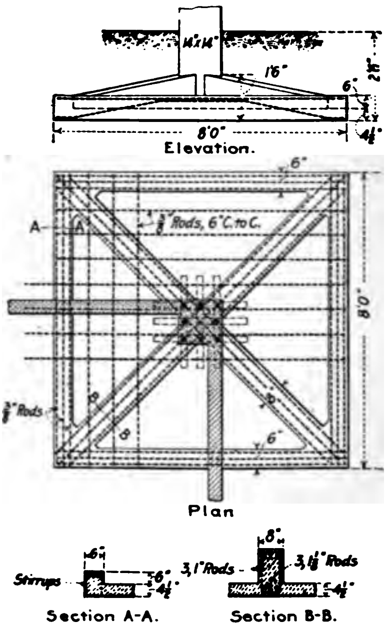


Fig. 219.—Column Footing for Retaining Wall, Atlanta Terminal Station.

to the sub-soil, otherwise it may settle unevenly and bending be brought upon the columns.

The spread footing used in the construction of the foundations for the Terminal Station in Atlanta, Ga., is shown in Fig. 218. As will be noted, this footing is modeled somewhat after the type used for the construction of cast-iron shoes. It has a bottom plate strengthened by cross ribs and the compensating flanges at the edge of the footing. This gives quite a shallow footing, with ample strength to secure the desired spread. Another footing of the same type is shown in Fig. 219. In this the shear is taken care of by the arrangement of the rods shown in elevation.

Figure 220 shows a reinforced concrete footing, for the Bush Terminal Building, described on another page. This footing, which is 11 x 12 ft. in size, was used as a capping for 18 timber

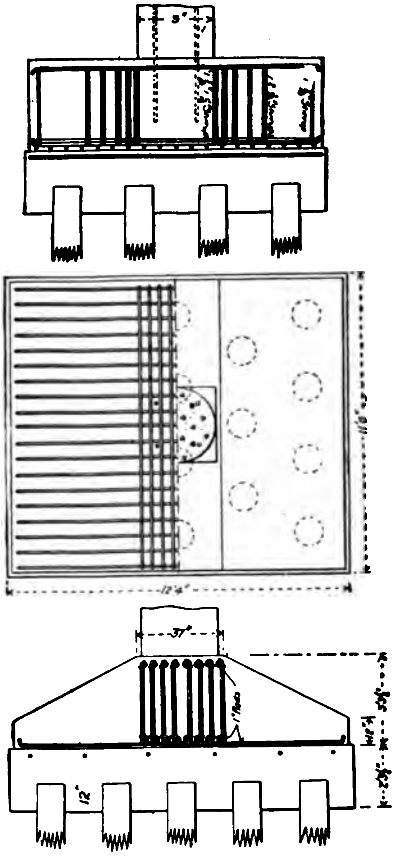


Fig. 220.—Column Footing, Bush Terminal Co. Factory.

piles driven about 18 ft. to a firm bearing, each pile being calculated to sustain a load of 20 tons. In the construction of this footing the lower portion of concrete capping having a thickness of 27½ ins., was placed around and over the heads of the piles. This part was reinforced with a single layer of 1-in. rods, spaced about 24 ins. center and located near its upper surface.

The upper portion, or the footing proper, has a thickness at the middle of 39½ ins., this thickness being reduced at the edges to 12 ins. Near the bottom surface are placed 1-in. transverse horizontal bars spaced 6¾ ins. centers. In a longitudinal direction the reinforcement consists of eight sets of 1-in. steel bars fastened together with vertical stirrups in accordance with the Bertine system. In the center of the footing four vertical bars 1 in. in diameter and 4½ ft. long are embedded in the footing

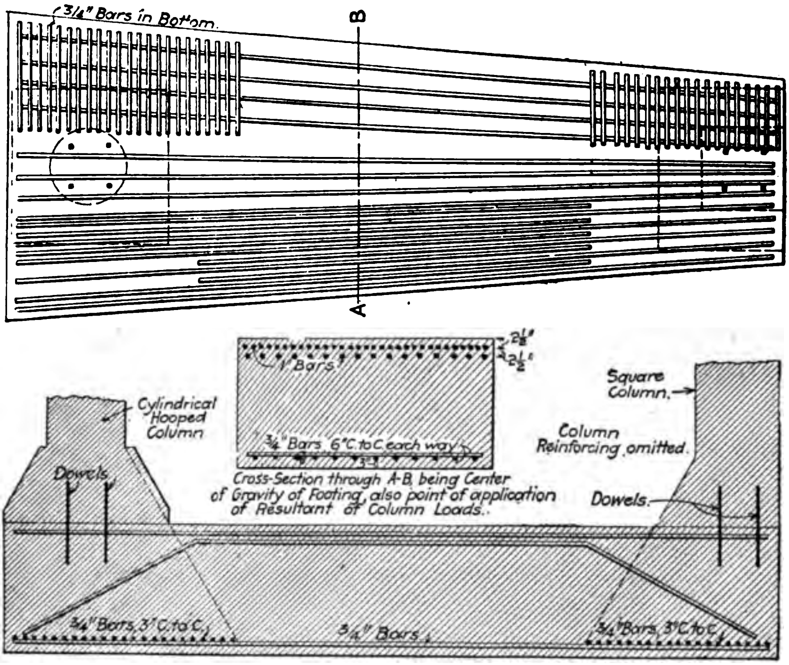


Fig. 221.—Combined Wall and Column Footing.

and extended above its upper surface to form dowels for connecting a column to the footing. These footings support the hooped columns described on page 474, and transmit to the soil through the medium of the piles some 360 tons.

A typical footing supporting a concrete column used in the Thompson and Norris eight-story factory building at Prince and Concord Sts., Brooklyn, N. Y., is shown in Fig. 287. Two or more columns are sometimes placed on a single footing. Figure 221 is an example of a spread footing supporting a wall and interior columns. Figure 222 shows a spread footing used to support a 400-ton smokestack and four columns in the same building. The dimensions, details and reinforcements used in both these examples are shown in the drawings. The type of reinforcement used was the Johnson corrugated bar. The supporting stratum is a hard, gravelly soil.

A special foundation of novel design is shown in Fig. 223.

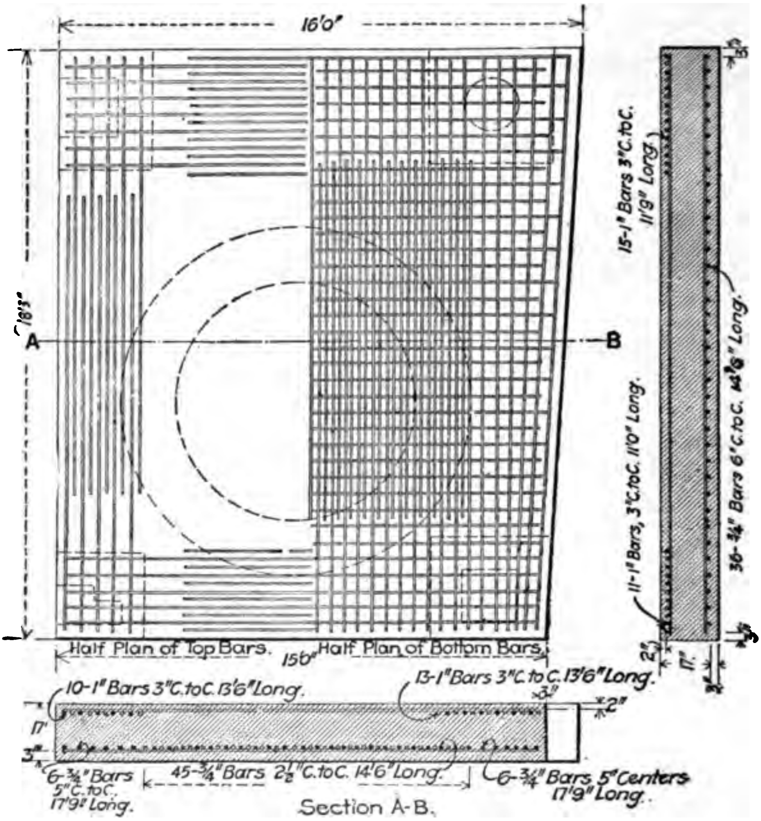


Fig. 222.—Footing for 400-Ton Smokestack.

This foundation was used for the Garage Building for the Decauville Automobile Co., New York City. The flaring shape given to the footing is for the purpose of distributing the loads uniformly to the soil, 1 ton per sq. ft. bearing being allowed on the soil. The amount of metal in each footing is given on the drawings. It should be noted that vertical shear bars are used where the columns come upon the footings; see cross sections. Figure 223 also shows a plan and section of a footing carrying four columns, designed for the same building, but not used, as rock

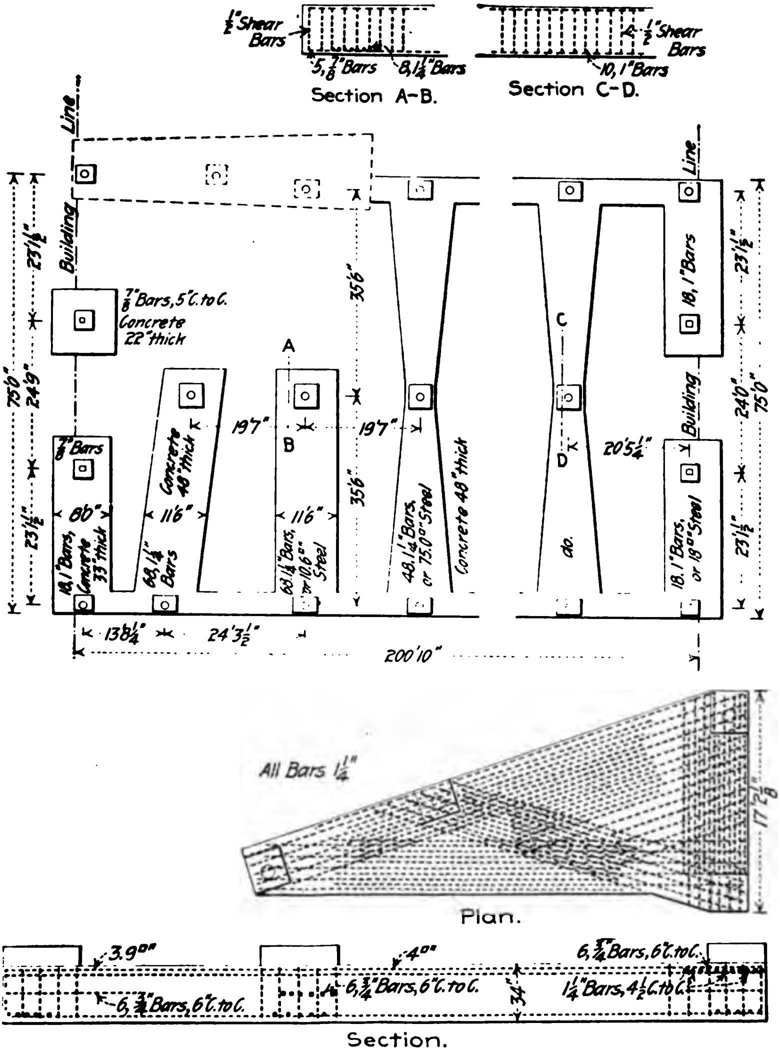
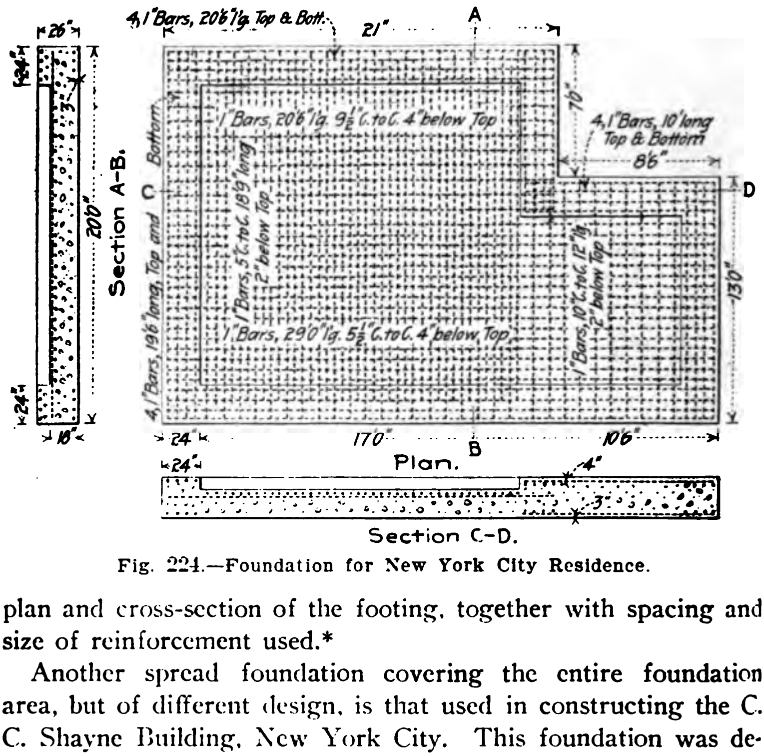


Fig. 223.—Foundation for Garage, Decauville Automobile Co., New York

was found under the part of the building where this footing was to have been used.

The footings are sometimes extended to cover the entire foundation area. An example of a small foundation of this character is that used in the construction of a four-story addition to a residence in New York City. The soil in this locality is filled in to a great depth, the filling consisting of earth and rock, the latter ranging in size from small stone to rocks of great size, and so poorly packed that local settlement is liable to occur at any time. A spread footing was used and so designed that the portions of the foundation under the walls will act as a cantilever beam or slab should any local settlement take place. Figure 224 shows a



Section C-D.

Fig. 224.—Foundation for New York City Residence.

plan and cross-section of the footing, together with spacing and size of reinforcement used.*

Another spread foundation covering the entire foundation area, but of different design, is that used in constructing the C. C. Shayne Building, New York City. This foundation was designed and built by the Hennebique Construction Co. It is of the ribbed slab type and spans the full width of the building. Details of construction are shown in Fig. 225. A 1 : 2½ : 5 concrete was used, the stone being trap rock, ½-in. and under.

Hennebique uses rod and stirrup reinforcements for column

*The writer is indebted to H. C. Miller & Co., of New York, for the designs shown in Figs. 223 and 224.

footings. Figure 226 shows a Hennebique column footing with this kind of reinforcement. This particular footing was used in a factory at Lisle, France, and carried a load of 130 metric tons. The reinforcement consisted of two courses of rods at right

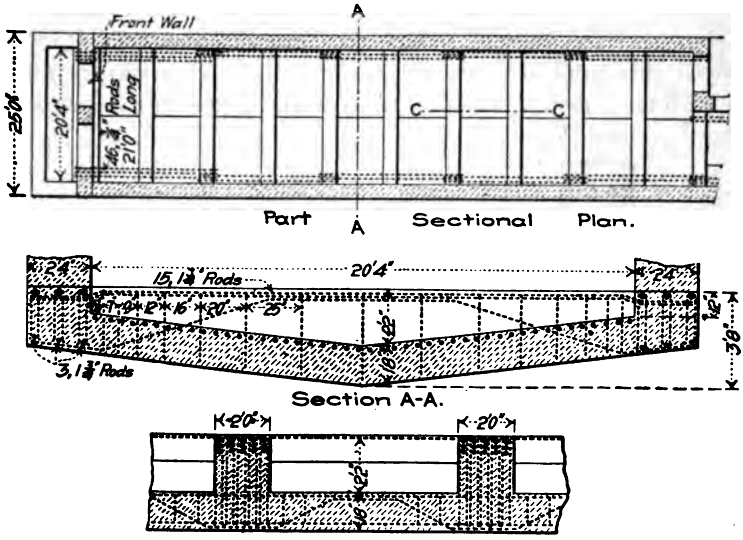


Fig. 225.—Foundation for C. C. Shayne Store, New York.

angles to each other, with stirrups extending upwards into the concrete from the lower tier of rods. In a later form of construction part of the round rods are replaced by flats, as shown in Fig. 227. For heavy foundations this engineer has used a series

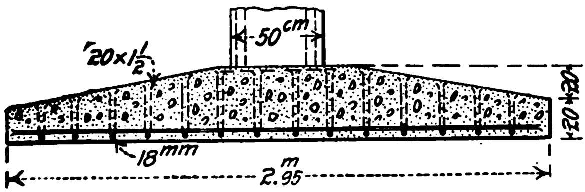


Fig. 226.—Column Footing with Bar and Stirrup Reinforcement (Hennebique).

of flats at right angles to each other and decreasing in length toward the top, and without stirrups. A layer of from 6 to 8 ins. of concrete is placed between each series of rods.

A special foundation which has proved efficient in soft ground is obtained by cutting up the foundation area with a series of intersecting brick or concrete walls having a covering slab of reinforced concrete. The series of bottomless boxes thus formed prevent the earth from spreading and the whole foundation acts together as a raft and is able to support heavy loads, although the ground may be very soft. The intersecting walls, if of concrete, may or may not be reinforced. M. Hennebique employs a system of this kind for spread foundation, using a modified flat ribbed floor slab with the ribs reinforced heavily on the compres-

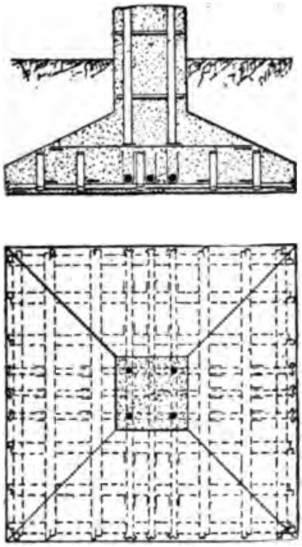


Fig. 227.—Footing by Hennebique, with Flat Bar Reinforcement.

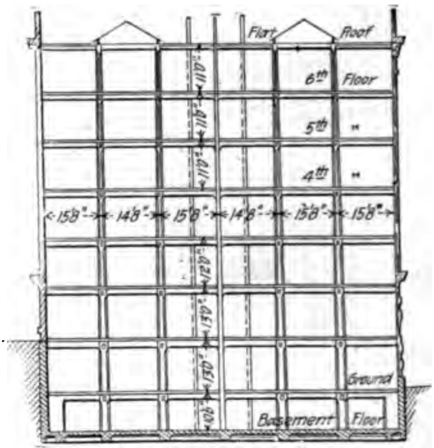


Fig. 228.—Section of Storehouse, Newcastle, England.

sion side as well as on the tension side. A warehouse for the Co-operative Wholesale Society at Newcastle-on-Tyne has a foundation of this character. This building is eight stories in height above the foundation and has a frontage of 92 ft. on the quay on which it abuts, and is 125 ft. deep. The floors are designed to carry 684 lbs. per sq. ft. The site upon which this building had to be constructed offered great difficulties for securing a suitable foundation. The first 18 ft. were of made ground, below this was 18 ft. of silt and quicksand, next came 10 ft. of soft clay, 5 ft. of hard clay and 10 ft. of silt and sand, and finally gravel. To add to this difficulty, the whole stratification had a decided dip toward the River Tyne. Piles could not be used on account of the danger of injuring neighboring property. The feasibility of sinking masonry piers to the gravel was considered, but owing to the uncertainty of ever securing a suitable foundation by this means and to the excessive cost, this plan was abandoned and a reinforced concrete raft was constructed. The whole building, as well as the foundation, was of Hennebique construction. Figures 228, 229 and 230 show the character of the construction. Each

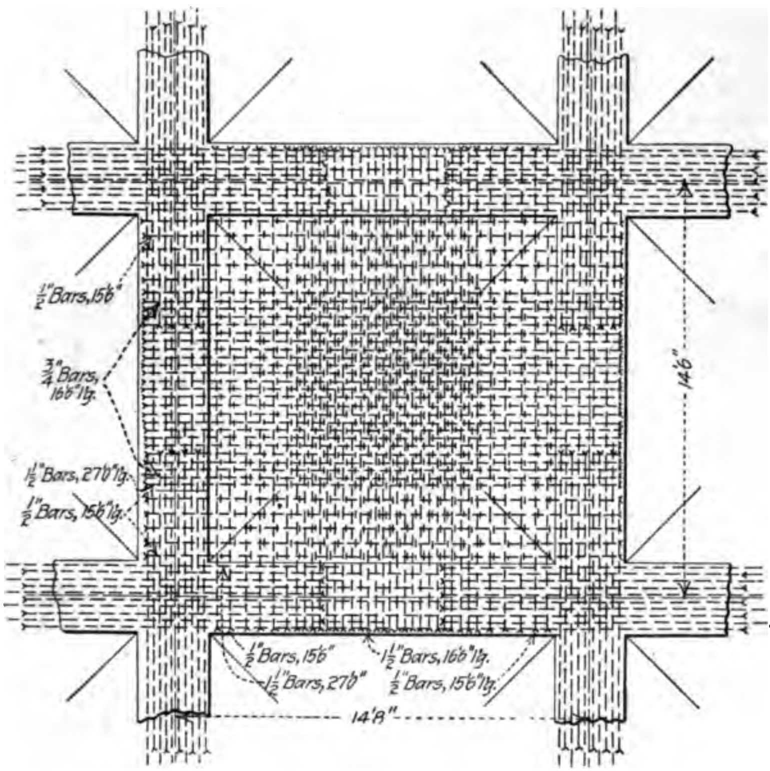


Fig. 229.—Plan of Floor Panel.

column rests on two intersecting beams 2 ft. 5 ins. wide by 2 ft. 6 ins. deep. These beams divide the area of the building into rectangular panels and in conjunction with the concrete floor arches, 7 ins. thick at the center, transmit the column loads over the whole area. A settlement of 3½ ins. at the front and 3 ins. at the back took place between the dates of the completion of the footing and the construction of the first floor. after which time no further settlement occurred.

The methods followed in the construction of reinforced concrete spread·foundations are similar to those outlined above for grillage beam footings. The ground must be carefully leveled off and the necessary boxing provided about the footing, and where ribbed foundations are used, as illustrated in the Hennebique construction for a warehouse at Newcastle-on-Tyne, trenches are dug for the ribs, and, if necessary, boxing provided. It is at times necessary to complete the forms with bottoms. The concrete is deposited in the forms or boxes in layers and the reinforcements put in place when the proper elevations are reached. If stirrups are used they are put in place and held upright by

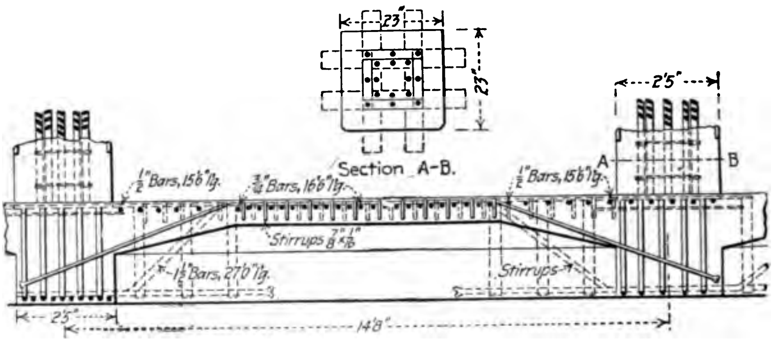


Fig. 230.—Section Through Floor Panel.

small mounds of concrete moulded about them, care being taken to have the stirrups bear closely upon the bottom of the rods. The concrete is then filled in until the level of the next reinforcement is reached; these are put in and concreting continued until the top is reached. The concrete should be carefully tamped as it is put in place. A coating of 1 to 2 cement mortar should be spread over the top of the footing. When large spread foundations are put in it is impossible to carry on the work continuously, and care must be taken in joining new concrete to old. The joint should be roughened and coated with rich mortar before beginning to lay concrete anew.

Pile Foundations with Reinforced Concrete Caps.—A thick mass of concrete placed about and resting upon the heads of timber piles has been extensively used in foundations for bridges, buildings, etc. By using reinforced concrete, a foundation is secured with as great stiffness and of much less weight than when masonry or unreinforced concrete is employed. A reduction in weight and a saving in cost is secured by the use of this form of construction. I-beams are sometimes used in a manner somewhat similar to grillage foundations. A thick layer of concrete is placed around and over the tops of the piles, one or two layers of I-beams are placed upon this concrete platform, and a filling of concrete is placed between and over these beams in the same manner as in spread foundations. Steel girders are sometimes used in a similar manner. A concrete slab reinforced with rods, some form of Monier netting, or with expanded metal is the more usual type of capping for piles.

A notable example of a pile foundation capped with reinforced

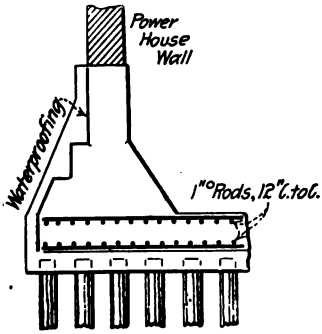


Fig. 231.—Capped Pile Foundation, Yonkers Power House, N. Y. C. & H. R. R. R.

concrete is that of the Yonkers Power House for the New York Central & Hudson River R. R. Figure 231 is a section near a side wall, and shows the concrete capping reinforced by two layers of 1-in. diameter rods. These layers are placed 22 ins. apart with two sets of rods at right angles to each other in each layer, and spaced 12 ins. centers. This foundation occupies an area of 265 × 265 ft., and is composed of timber piles driven 3 ft. centers and capped with concrete reinforced as explained.

When expanded metal is used it is fastened to each pile by a staple. Figure 232 shows an expanded metal reinforcement used in a pile foundation for the masonry piers for the Penrose Ferry Bridge over the Schuylkill River at Philadelphia, Pa. No. 4 gauge plate, with 6-in. mesh, was used in this construction.

In the construction of reinforced concrete pile caps the earth is excavated to the desired depth below the top of the piles, they having been cut off at the proper elevation. The concrete is then filled in around and over the piles to the level of the reinforcement, being confined, if necessary, at the sides with boxing. The reinforcement is then put in and the concreting continued until the thickness desired is secured.

Concrete Piles.—Piles constructed of concrete may or may not be reinforced. They can, in general, be used under almost all conditions in which timber piles are employed, and in many situations where the latter would be impracticable. They may be used in dry soil, in soil which is wet and may dry out or is alternately wet and dry. Wooden piles will decay when subjected to alternately wet and dry conditions. On this account timber piles

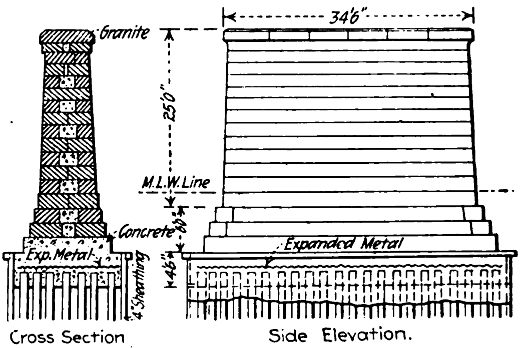


Fig. 232.—Capped Pile Foundation, Penrose Ferry Bridge, Philadelphia, Pa.

must be cut off below the water line, necessitating often a large amount of concrete as a filling to bring the foundation up to the desired level. By the use of concrete piles capped with reinforced concrete much less material will be needed. A saving is also made in the amount of excavation.

Concrete piles cost somewhat more than timber piles, but under many circumstances the saving in materials and excavation when they are used is great enough to make a substantial saving in the total cost of the foundation.

Difficulties in driving have been experienced in some soils, but by the use of improved methods it is probable they can be driven in almost any soil that timber piles will penetrate. Concrete piles may often be used when it is difficult to secure those of timber. Piles of this kind also possess the advantage of being readily bonded to the capping of concrete or concrete-steel. They possess marked advantage over timber piles for marine work on account of their freedom from the ravages of the teredo and their independence from the influences of moisture conditions. When used as reinforced sheet piling grooves are left in adjacent edges of the piles; these are filled with grout after the piles are driven, and thus become practically a watertight monolithic wall possessing great strength.

Concrete piles may be divided into two general classes—those constructed in place and those cast or moulded in advance and driven after the concrete has attained sufficient strength by methods similar to those used for driving timber piles.

Piles Built in Place.—Three forms of concrete piles built in place deserve consideration. The methods of construction employed in building them differ somewhat.

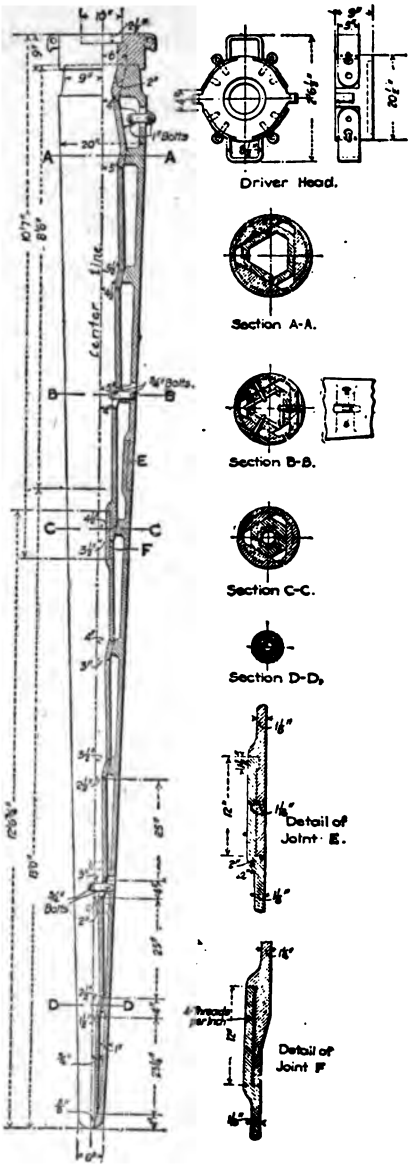


Fig. 233.—Collapsible Core, Raymond Concrete Piles.

The Raymond Pile.—The Raymond Concrete Pile, the patents of which are controlled by the Raymond Concrete Pile Co., of Chicago, Ill., is constructed as follows: A steel pile-core, the size and shape of the pile desired, is encased in a thin, closely fitting steel shell, and driven into the ground by means of a pile-driver in the usual manner. The core is then withdrawn, leaving the shell in the ground. The shell has sufficient strength to retain the shape of the core. This hollow tube is then filled with concrete. When reinforcement is to be used it is placed before putting in the concrete. The details of the core, which is collapsible, to facilitate drawing, are shown in Fig. 233.

This particular core is adapted to the construction of piles 30 ft. long, 6 ins. in diameter at the point and tapering to 20 ins. in diameter at the top, and weighs 7,500 lbs. The core consists of a hollow tapered center-piece and a conical shell divided longitudinally into three segments with a small clearance between their vertical edges. The center-piece is made in two sections, connected by a screw joint (see detail F, Fig. 233), and has a surface composed of vertical faces connected by short inclined sections or bearing surfaces, which fit and bear against wedge-shaped projections on the inside of the shell segments. Ample clearance is thus obtained between different parts of the core; this allows the shell to move a short distance up or down with relation to the center-piece and allows it to expand or contract, as may be required. The outer segments are also made in two pieces and fastened together with a lap joint riveted with countersunk rivets. The center-piece and segmental shell are fastened together with horizontal links at the bottom, top and intermediate points.

The conical shape given to the pile helps to compact the earth when driven and gives a greater bearing or frictional area than is obtained by the use of a straight pile. When the piles are driven to bed-rock a special core, 13 ins. in diameter at the bottom and 20 ins. at the top, is employed. These piles form concrete columns, upon which the substructure is built. Bent steel plates are tap-bolted to the top of the core to form guides to engage the pile-driver ways. The head of the core is recessed to receive a hooped oak driving block. The mould, of No. 20 gauge, one-pass, cold-rolled Bessemer steel, is prepared in 8-ft. sections, which overlap several inches. The steel is shaped in an extra-heavy cornice break. The sections are nested and placed on the point of the core. The upper sections are then pulled into place and fit closely about the core. The core is then lifted in the ways, centered as required and driven like an ordinary timber pile.

After having been driven to the proper depth, the key which joins the center and shell of the core is removed, a tackle hitched to the center of the core, and as tension is put upon it the core collapses, is lifted out and prepared for the next pile. Fig. 234 shows both the core and the shell.

When it is desired to strengthen the pile against flexure, the re-

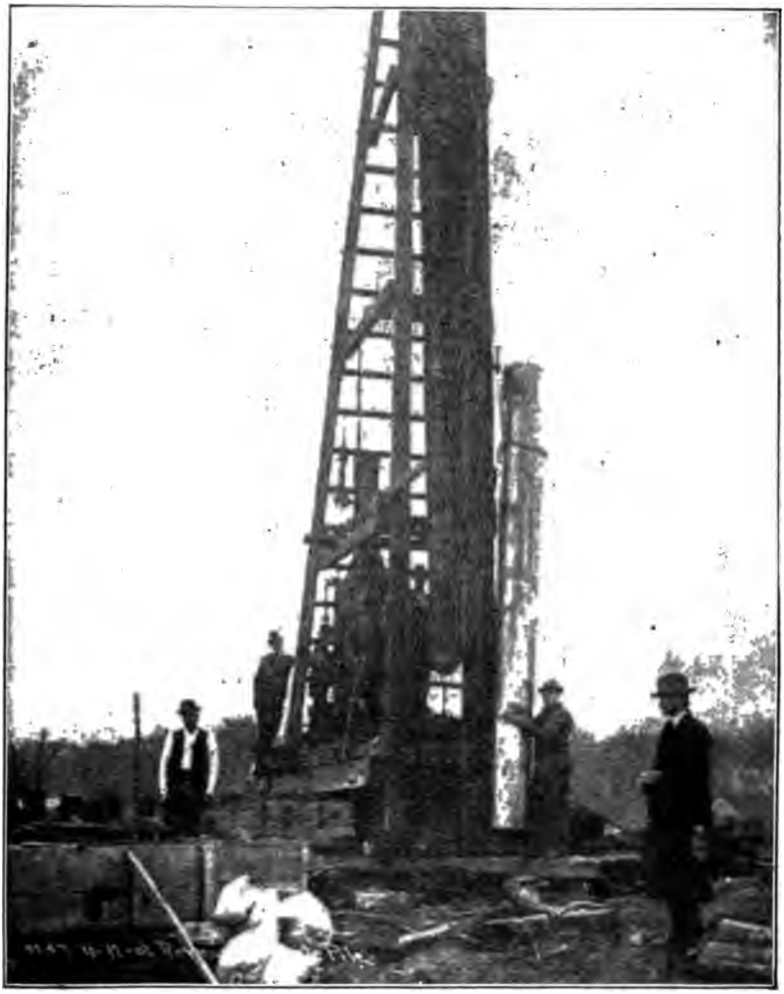


Fig. 234.—Core and Shell of Raymond Concrete Pile.

inforcement is placed before putting in the concrete. This usually consists of a central rod about $1\frac{1}{2}$ ins. in diameter and three $\frac{3}{4}$-in. side rods, placed as shown in Fig. 235. After placing the rods the concrete, which is usually 1 : 2 : 4, is put in. It is considered that no ramming is needed in the lower 10 ft., but the remainder of the concrete is carefully rammed as it is placed.

An ordinary pile-driver, slightly modified, may be used for driving the Raymond pile-core, but on account of its great weight a heavier hammer should be used than is employed in driving timber piles. A special machine, as shown in Fig. 236, has proved more convenient and rapid than the ordinary pile-driver. This machine has a 50-ft. tower, with telescopic extension for handling long piles or operating a hammer in a trench. A Vulcan steam hammer is used with this machine, which strikes from 60 to 70 blows a minute. With this machine an average of 40 piles a day should be driven.

Raymond piles have been successfully used for many structures. One hundred and forty-two piles, 20 ins. at the top and 13 ins. at the bottom, with a shell of No. 20 sheet iron, were used in the construction of a library building at Aurora, Ill. These were

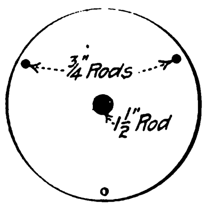


Fig. 235.—Reinforcement for Raymond Concrete Pile.

driven 14 ft. through filled ground to rock by an ordinary pile-driver with a hammer weighing 2,400 lbs. and having a drop of from 20 to 25 ft. In the construction of a receiving station for the United States Express Co. in New York 163 Raymond piles were used. These piles are 6 ins. in diameter at the bottom and 20 ins. at the top and range from 22 to 28 ft. in length. The soil in this locality is a deep, soft mud or silt up to ground water line; above this there is a 10-ft. fill containing a large quantity of ashes and broken stone. It proved a difficult soil to drive through. A pile-driver of the ordinary type, with a 3,200-lb. hammer, was used, with a maximum drop of 10 ft., and only averaged 13 piles in a 10-hour day.

This pile is adapted for use when the soil to be penetrated is sand, quicksand or silt or soft earth, which may be loosened and removed by a water jet. The pile has a sheet-iron tapering shell made in 8-ft. sections and nested (Fig. 237). Each section has on its upper end an outside projecting ring, which engages a similar ring on the lower end of the section above and pulls it down as it sinks. A cast-iron shoe is riveted to the inner and lower section and a 2½-in. pipe with a ¾-in. nozzle is attached

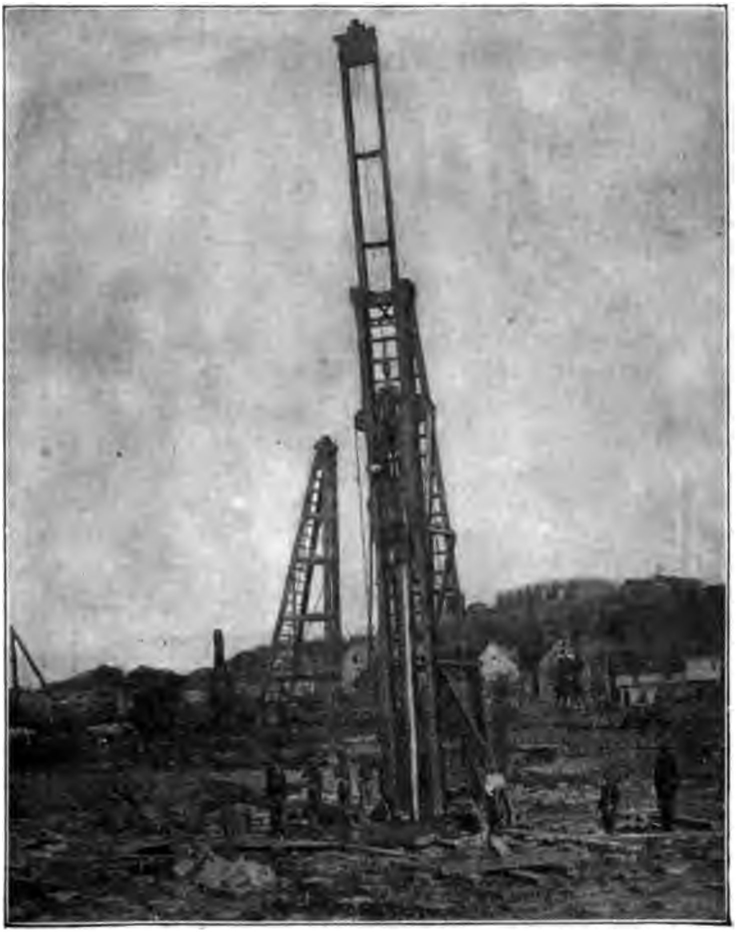


Fig. 236.—Pile Driver for Raymond Concrete Pile Work.

to the center of the pile point or shoe. This pipe is held in the center of the shell by spreaders placed at each joint. Water under pressure is forced through this pipe; the first section sinks until its head engages the bottom of the second shell and draws it down, and so on. Each section is filled with concrete as it sinks, and additional sections are added to the pipe as is necessary. The 2½-in. pipe is left in the center of the pile and acts as a reinforcement. If additional reinforcing is desired, rods may be inserted near the outer face of the concrete. During a test of this system on the Missouri River near Omaha, Neb., a pile of

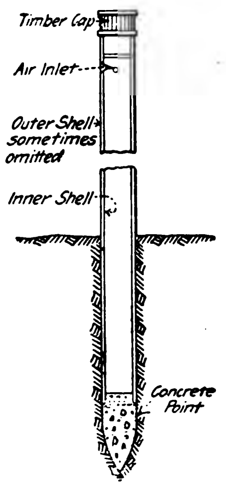


Fig. 238.—Simplex Pile Driving Form, Point and Cap.

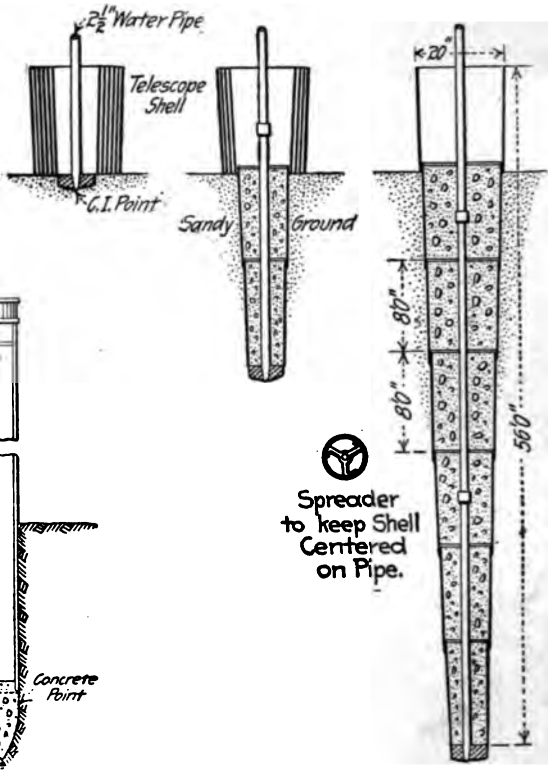


Fig. 237.—Telescopic Shell for Raymond Concrete Pile.

this kind, 10 ins. in diameter at the bottom and 20 ins. at the top, was sunk to a depth of 75 ft. in sand with a water pressure of only 40 lbs. on the jet.

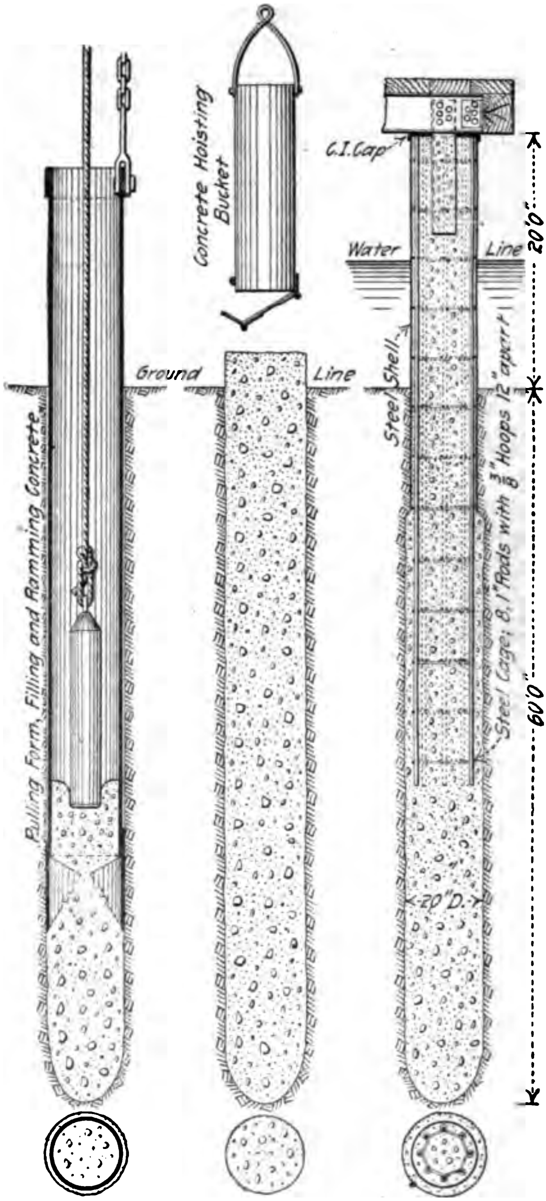
The Simplex Pile.—The Simplex Concrete Piling Co., of Philadelphia, Pa., and the Foundation Co., of New York, employ a wrought-iron driving pipe of the diameter and length of the pile required. This pipe is of sufficient strength to stand driving, and has a point made of concrete or of cast iron or steel, and is capped with a hardwood driving head which protects the pile from being injured during driving. After being driven to a firm bearing the pipe is withdrawn and the hole filled with concrete. An ordinary pile-driver is used for driving this pipe. Figure 238 shows section of the Simplex driving form. In certain kinds of soil, what is known as the alligator point is used. This consists of a folding point which opens and is withdrawn when the pipe is drawn. The character of this point is clearly shown in Figs. 239 and 245. Fig. 239 shows shell and point being withdrawn and concrete being tamped in place; Fig. 240 a section of a finished pile, together with the concrete bucket; and Fig. 241 shows method of reinforcing pile when used for wharves and docks. Figure 242 shows concrete points used in the construction of the Pittsburg Terminal Warehouse, while Fig. 243 shows cast-iron points used for certain classes of soils.

In the construction of foundations for the Washington Barracks some modifications of the above plan were found necessary. Capt. John S. Sewell, Corps of Engineers, U. S. A., who had charge of this work, gives the following information regarding it:

It was found that a concrete point having its main body 4 ins. greater in external diameter than the driving pipe and shaped like an elongated projectile was the most satisfactory. A tenon was formed on the upper end of the point, which fitted into the lower end of the driving pipe. The body near the tenon tapered slightly inward. The shoulder where the tenon joined the point was reinforced by a circular ring of cast iron, and the body of the point below was reinforced by a cylindrical wrapping of expanded metal placed near the surface to resist the bursting effect due to driving.

The driving form consisted of a heavy cast-iron driving pipe fitted over the tenon of the point. Enveloping this was a wrought iron pipe of larger diameter, which fitted firmly on the sloping part of the body of the point. At the top was a hardwood driving head with a tenon fitting into the heavy pipe and having a shoulder broad enough to rest upon both pipes, and fitted with projections to keep the piles properly centered.

This composite form was set up and driven to a firm bearing with an ordinary pile-driver having a hammer weighing 2,200 lbs. The driving pipe was then lifted out with a rope and tackle.



Figs. 239-241.—Sketches Showing Construction of Simplex Piles.

After three or four shells were driven, the filling and drawing of the outer pipe was begun. The pulling was done by means of a tripod and winch. The concrete was put in and rammed as the outer pipes were pulled, so as to keep the bottom of the pipe always sealed with a foot or more of concrete. In this manner the hole was entirely filled without any danger of caving in.

This method can be applied to piles driven through water, the outer cylinder being left in place. It was found that through moderately wet earth the outer pipe could be dispensed with, and the plan finally adopted under such conditions at Washington

Fig. 242.—Concrete Points for Simplex Piles.

Barracks was as follows: A driving pipe about 1 in. less in diameter than the point was used. The sections of the pipe are spliced with outside sleeves, as any projection within prevents the proper placing of the concrete. The pipe was also reinforced with sleeves at the bottom and top. These sleeves do not add materially to the difficulties of driving or drawing. The friction in driving was greatly reduced by coating the outside of the pipe with grease or heavy oil. When difficult driving is encountered, steel points should be used.

An attempt was made to reinforce the piles with circumferential cylinders of expanded metal, but the dropping and ramming of the concrete caused these to collapse, so that the attempt failed.

A center rod was successfully used for reinforcement at Washington Barracks. In work of more recent date the reinforcing has been accomplished by using four or more steel rods connected by a system of horizontal wires, the rods being placed near the circumference of the pile.

Recent work done by this method has resulted in the elimination of the interior cast iron form, and the so-called Alligator point is now generally used in place of the cast iron or concrete points.

A specially designed pile-driver is used, which has an in-

Fig. 243.—Cast Iron Points for Simplex Piles.

genious arrangement by which the steel driving form can be pulled without moving the pile-driver. A pair of pulling ropes is passed between the hammer and the leads, a place being provided for this purpose by using a steel channel to form the inner face of the lead. The flanges of the channels are turned toward the hammer, and form a pocket or channel for the pulling ropes as well as acting as an inner face for the pile driver leads. By the use of this form of driver the entire operation is performed with one outfit, and the pile form is pulled immediately after driving, the concrete being placed in position at the same time. In cases where the surrounding material tends to fill the void left by the driving pipe when it is withdrawn, a light cylindrical shell of No. 20 or 22 gauge metal, having an outside diameter slightly less than the inside diameter of the driving form, is dropped inside of the form when the latter has reached its final depth. This shell is filled with concrete before withdrawing the driving form, and serves to retain the form of the pile and to reduce the friction of drawing by relieving it from the internal friction of the concrete. By this method Simplex piles up to 40 ft. in length have been driven. The power required to draw the form is often very considerable. It is stated that in one case an uplift of approximately 100 tons was employed. This throws an interesting light on the probable friction of the soil against a pile.

One of the advantages claimed for the Simplex pile is that the steel shell employed for the outer form is light, and does not absorb the shock of the pile-driver. The inertia of the light shell being small, it does not reduce the penetration, and on this account it is claimed that this pile may be driven to a much greater depth in hard driving than piles driven by other methods. It has been found that this pile works best in its simplest form in clay soils, but with the addition of the light inner shell it can be used in soft soils, quicksand, and practically any materials, even in water.

The cost of driving per lineal foot for long piles is somewhat greater than that of driving short piles, but the cost of the point, which is an important item in the cost of the pile, brings the total cost per foot of short piles up to about that of long ones. The cost of driving alone is somewhat higher than that of driving timber piles; to this must be added the cost of drawing and filling the tubes, which is considerable. The cost of steel points is about 60 per cent. more than that of concrete points.

The Barracks buildings were located on an irregular strip of land between the Potomac River on the west and a former tributary, known as the James Creek, on the east. Much of the site is reclaimed land. On the Potomac River side the original river-bed was good firm sand and gravel, with many boulders. The bottom of the James Creek was soft silt for a depth varying from 25 to 45 ft. below mean low water. The present surface of the ground is from 10 to 18 ft. above low water. The permanent water line is as low as mean low water everywhere. Some of the made earth was compact, but even the best of it would not safely carry a pressure of 500 lbs. per sq. ft. Timber piles were out of the question in this locality. Concrete piles varying in length from 10 to 45 ft. were successfully used.

One of the advantages claimed for this pile is the great surface friction developed by the rough surface of the concrete. The vigorous ramming of the concrete as it is put in place causes it to expand into the surrounding earth, compacting the latter, as well as causing an intimate contact between the two, and leaving a rough surface to the concrete, which develops high frictional values. Usually a rammer weighing 6,000 lbs. is used.

These piles may be used in all kinds of ground; hard ground, filled ground, filling of ashes or other materials, and swamp or other soft ground. They are practically indestructible after once in place. There appears to be no limit to the length of pile which may be driven, while the length of a wood pile is limited by the height of the pile-driver frame; in the case of the Simplex pile the pipe may be put on in sections and the driving continued until the desired depth is reached. Again, as a metal pipe has greater strength than a wooden pile, it may be driven through ground so hard that it cannot be penetrated by a wooden pile.

The alligator point has been successfully used in alluvial soils in the South. It was used in the construction of the foundation for a mill building at Holyoke, Mass. This building is 300 ft. × 60 ft., two stories in height, and was built on a river bank. The soil was filled in and consisted of sand and mud. The piles averaged 23 ft. in length. The alligator point was also used in the construction of a foundation for a warehouse for the Belknap Hardware Mfg. Co., at Kansas City, Mo.

Simplex piles were used for the foundation of the Produce Exchange Bank Building in New York City. This is a 12-story and basement building; 125 piles, averaging 20 ft. in length, were driven through quicksand into hardpan. The piles were loaded to 35 tons each. A test of the bearing power of a pile was made under the supervision of the Bureau of Buildings. When loaded to 45 tons no settlement was observed. A test was made of five 16-in. piles for a crane foundation for the Westinghouse Machine Co., at Pittsburg, Pa. This test continued for ten days. No settlement was observed under a test load of 300 tons.

Simplex piles can be put in with great rapidity, anywhere from

20 to 50 piles being put in with a single pile-driver in one day. The foundation for the new River and Railroad Terminal Warehouse Building, at Pittsburg, Pa., is constructed on 4,800 16-in. Simplex piles, from 30 to 45 ft. long, driven through filled in ground. The actual time of driving was 76 working days.

Figure 244 shows a chimney foundation constructed of Simplex piles; 12 piles, driven about the circumference of a circle, were used. Anchor rods extending to the bottom of the piles prevent any possibility of overturning from wind pressure. This

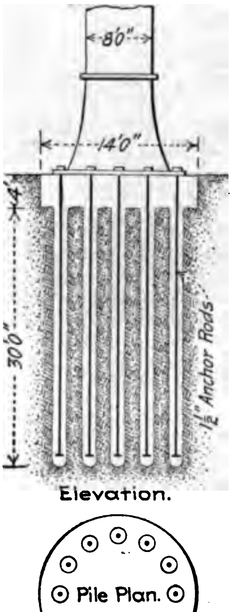


Fig. 244.—Chimney Foundation of Simplex Concrete Piles.

particular foundation was used in the construction of a chimney at Holyoke, Mass.

Figure 245 shows a pile-driver and driving form with alligator point attached, used at Sioux City, Ia., in driving Simplex concrete piles for the C., St. P. & O. R. R. Co. round house. This foundation was constructed by the Foundation Co., 35 Nassau St., New York.

When it is desired to use a heavy reinforced pile for wharf construction, the construction shown in Fig. 241 may be used. After filling the lower part of the pile with concrete, a reinforcement, fastened together as shown, and surounded by a sheet-iron shell to hold the concrete in place in the part passing through the water, is lowered in place and concrete filled in to the top.

Fig. 245.—Simplex Pile Driver and Alligator Point Driving Form.

Steel-Concrete Pile.—A pile using a steel shell is extensively used by Clark & Co., New York City, for building foundations. The shell is of seamless steel, ⅜-in. or more in thickness, and is used in sections of convenient length. The sections are joined by an inside coupling (see Fig. 246). The pile is driven by means of a steam hammer and water jet. Figure 247 shows the arrangement of the apparatus for driving. After the shell has reached rock and the confined material has all been washed out, which is shown by the water at D running clear, then by changing the

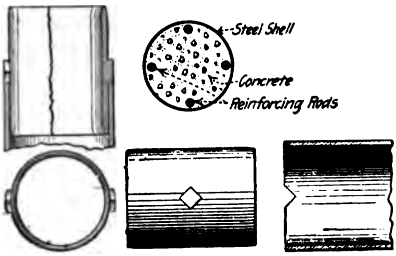


Fig. 246.—Clark Pile and Driving Shell.

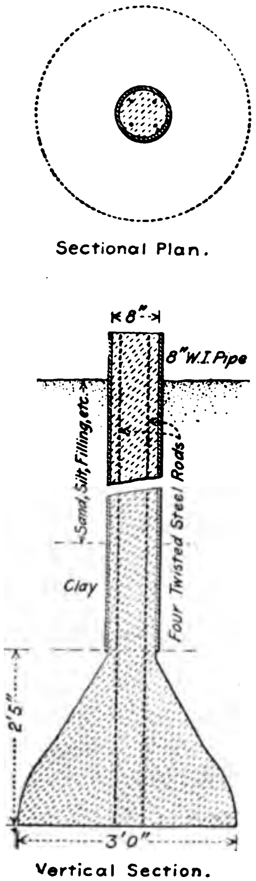


Fig. 248.—Reinforced Concrete Pile with Enlarged Footing.

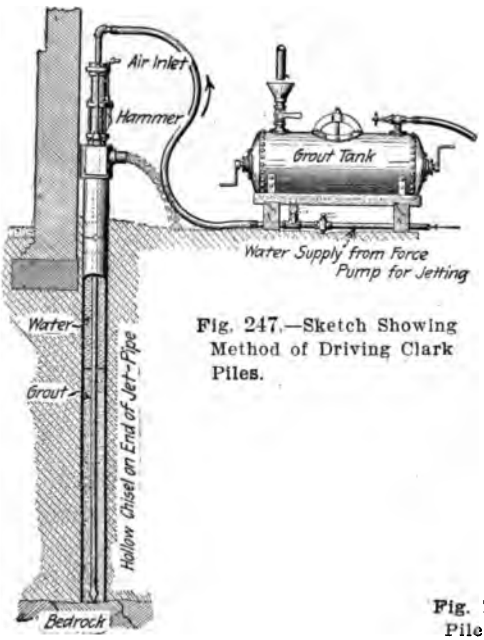
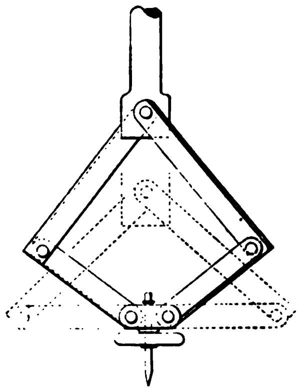


Fig. 247.—Sketch Showing Method of Driving Clark Piles.

valves grout is forced down the jet pipe, which by its greater specific gravity replaces the water. A neat cement grout is used to fill the pipe. When reinforcement is desired, round rods, held in proper position by rings, are used about the periphery of the pile core.

Reinforced Concrete Piles with Enlarged Footing.—Figure 248 shows a reinforced concrete pile with an enlarged footing, used for a foundation in clay in underpinning a building in Boston, Mass. The total length of this pile was 26 ft. It was constructed by driving a tube to a depth somewhat less than that desired for the footing, then boring and washing out the hole to the full depth required, which is about 2.5 ft. below the bottom of the tube. The lower part is then chamfered out with the machine shown in Fig. 249. The angles forming the cutting edge of the machine are sharpened to knife edges, and by turning the machine from above the hole was bored out, the material being



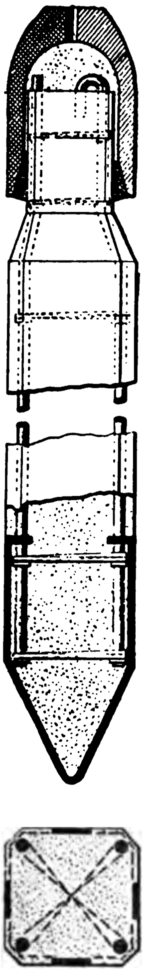
Chambering Machine.

Fig. 249.—Chambering Machine for Constructing Pile with Enlarged Footing.

washed out as the boring proceeds. The pin at the bottom of the machine prevents side motion and holds the machine in line during the process of boring.

Reinforced Concrete Piles Moulded Before Driving.—Piles moulded in advance resemble columns, both in the arrangement of the reinforcement and the manner of moulding. The reinforcements used consist of one or more rods or shapes placed either on the axis of the piece or near its perimeter.

Hennebique was the first engineer to use reinforced concrete piles. He first made use of them in 1896, and has since extensively employed them, both as bearing and sheet piling. Figure 250 shows a simple and common type of Hennebique pile. Usually four reinforcing rods are employed, although sometimes eight are used. In the latter case a rod is placed at the middle of each de. The reinforcing rods are tied together at intervals of about ins. with iron wire ties. The point is strengthened with an iron ıoe. The pile shown in section by Fig. 251 was used in the con-



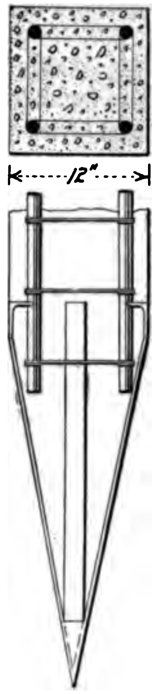
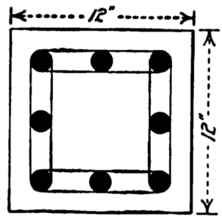


Fig. 251.—Pile for Dock Work at Southampton, England.



ig. 250.—Hennebique Type of Pile, Showing Driving Cap.

Fig. 252.—Section of Pile for Quay Wall at Southampton, England.

ruction of a dock at Southampton, England. Figure 252 is the ction of a pile used in a quay wall at the same place.

The head of the pile is sometimes moulded round and to a naller diameter than the thickness of the body of the pile, to permit the application of a driving cap. Figure 250 shows the type of cap employed when this form of head is used. The rods sometimes extend beyond the concrete and are bent into a loop, to which a tackle may be hooked for lifting them.

Figure 253 shows a hollow Hennebique pile, which is much lighter than the ordinary pile, and hence more easily handled. The four reinforcing rods are retained, and the diaphragms, which contain forked spacers and wire ties, join them together.

The Hennebique sheet piling is constructed in the form of rectangular slabs and is usually reinforced with six vertical rods, two being placed in the middle of the slab. The rods are tied together in both directions in the horizontal plane. A semicircular groove is formed down the edge of each pile and the hole formed by two of these grooves, when the piles are driven side by side, is filled with grout. The pile point is formed by beveling one of the narrow faces so that it forms a wedge with the opposite face. Figure 254 is a section of Hennebique sheet piling used in the construction of a bank protection on the Ghent Terneuzen Canal, Belgium.

An example of a square pile of the Hennebique type, but of American design, is shown in Fig. 255. This pile was used in the construction of the Railway Terminal Station at Atlanta, Ga.

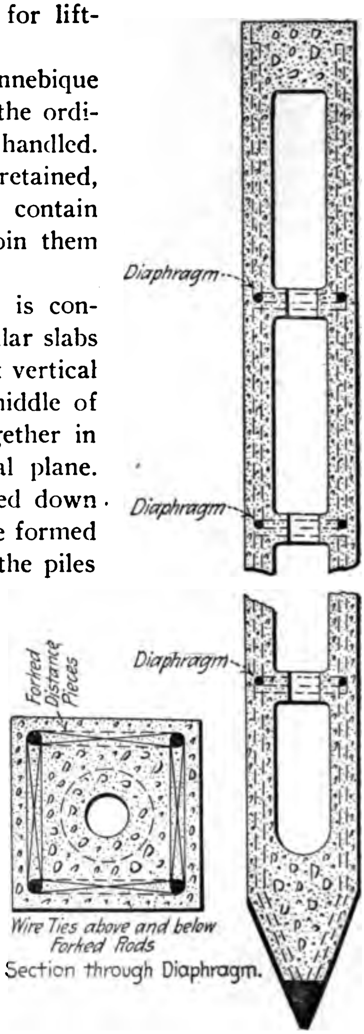


Fig. 253.—Hollow Pile of Hennebique Type.

Four reinforcing rods, $1\frac{1}{4}$ in. in diameter, are used. These are bound together by $^3/_{16}$ in. wire ties, spaced nearer together than in column construction. The tops of the reinforcing rods generally extend to within about 2 ins. of the top of the concrete at the head. The point is formed by an ordinary pile shoe, the straps of which are turned in at the top, to form a hold in the concrete. The reinforcing rods are bent in at the foot and bear against the bottom of the shoe.

Figure 256 shows the form of a rectangular pile used in the construction of a wharf at Novorossisk, Russia. The piles were spaced 16 ft. 4¾ in. centers and supported 15¾ × 25⅜ in. con-

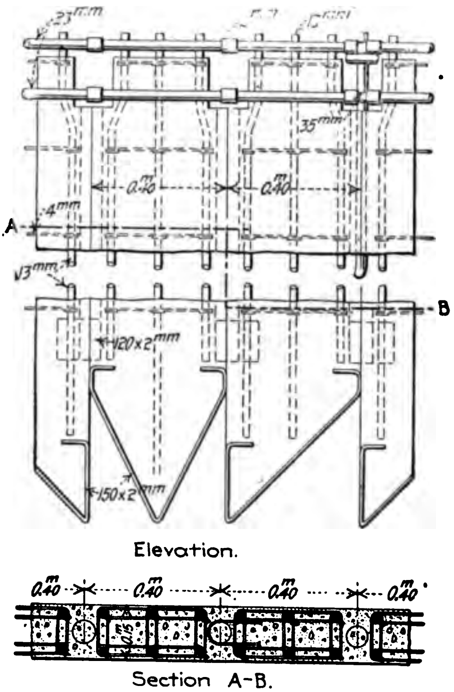


Fig. 254.—Sheet Piles of Hennebique Construction.

crete steel girders. These were connected by five 8 in. × 12 in. stringers, which in turn supported a 4-in. reinforced concrete slab. The load to be carried was 213 lbs. per sq. ft., each pile carrying the load from 269 sq. ft. of floor, consisting of a total dead and live load of about 43 tons. These piles were approximately 16 ins. square and 42 ft. long, and have a reinforcement weighing about 45 lbs. per lin. ft.

Figure 257 shows the section of a triangular pile used in the

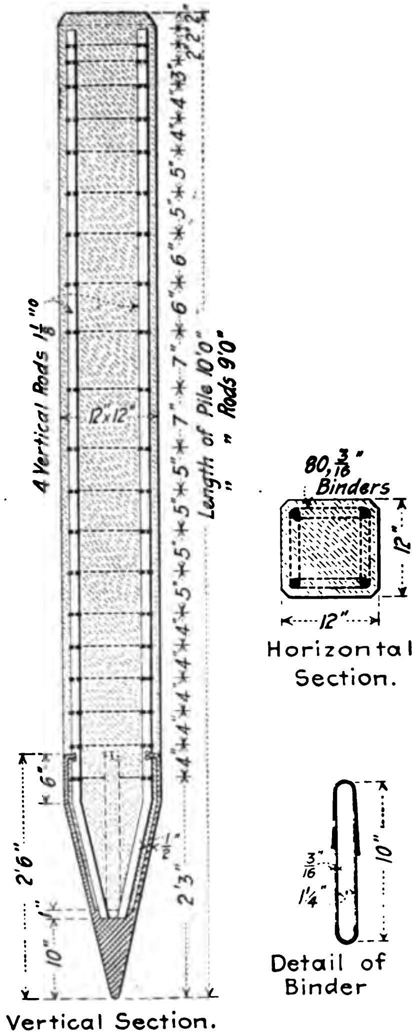


Fig. 255.—Pile for Trainshed Foundation, Atlanta, Ga., Railway Terminal Station.

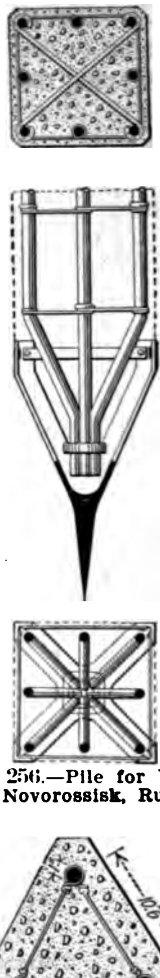


Fig. 256.—Pile for Wharf at Novorossisk, Russia.

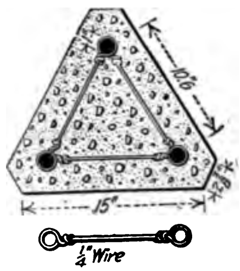


Fig. 257.—Pile for Court House Foundation at Berlin, Germany.

construction of a courthouse foundation near Berlin, Germany. The reinforcing rods are 1-in. in diameter and are tied together at intervals of 8 ins. throughout their length with ¼-in. ties. The rods are bent to a point and welded together at the bottom. The concrete used was 1 part Portland cement to 3 parts river gravel.

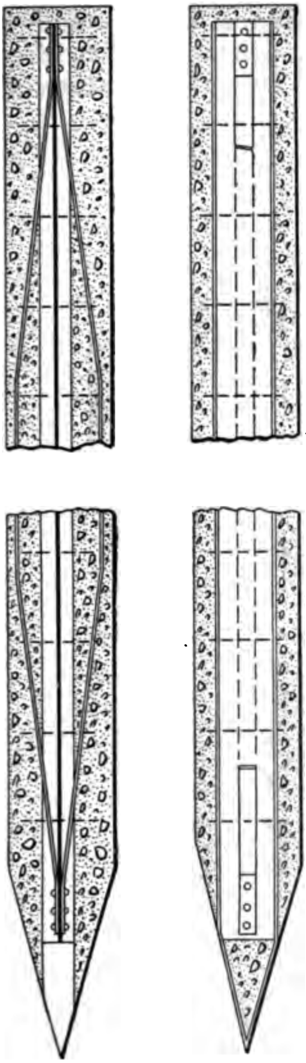


Fig. 258.—Williams Pile.

The form of pile reinforcement shown in Fig. 258 is the invention of Mr. A. E. Williams, Inst. C. E., of Dagenham Docks, Essex, England. The reinforcement consists of an I-beam. At the point the web is cut away and the sides forged to a point. The concrete is further reinforced by steel hoops placed at short distances apart throughout its length. When bending in a direction normal to the web of the beam is feared, two flat steel bars are attached to the web at the ends and bent out in the form of a truss. A pile shoe of the ordinary form may be used, but is sometimes omitted.

Another form of I-beam reinforcement, described by Christophe, in Beton Armé, is called the Rechter, Vereng and Dopking system, and is adapted to the construction of sheet piling. The reinforcement consists of two I-beams, tied together at intervals by framed diaphragms. A tongue is moulded on one edge and a groove on the other which lock together adjacent piles when they are driven. This pile was designed for a quay wall at Cochin-China, and has an anchor rod attached to the top and carried back into the bank, where it is rigidly anchored (Fig. 259).

The author knows of few cases where round reinforced piles have been used. Presumably the reason is on account of the additional cost necessary to construct forms for circular sections. This reason seems more imaginary than real, for if a considerable number of piles are to be used, by careful designing circular moulds can be constructed at a cost little, if any, greater than

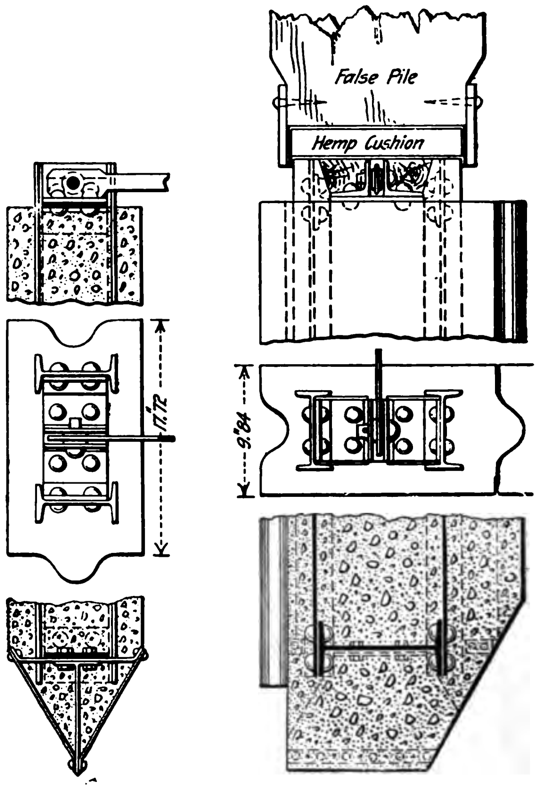


Fig. 259.—Pile for Quay Wall at Cochin China.

those of square or rectangular section. With the use of circular moulds the hooped reinforcement used in columns could be employed to replace the ordinary form of longitudinal rods with light rods placed about the circumference to take care of any possible bending stresses. The great ductility possessed by hooped concrete should enable it to resist, without injury, the shocks due to driving. The concrete is often greatly injured at the head, and sometimes throughout the pile, when reinforced in the ordinary manner, by the shock of the hammer when the pile is driven.

A reinforced concrete pile recently designed by Mr. Frank B. Gilbreth is called the Corrugated Reinforced Concrete Pile. The character of this pile will be understood from the cross-section shown in Fig. 260.

The piles taper uniformly from the butt to the point. The piles used for the foundation of the Lattemann Building, in Brooklyn, N. Y., were 15 ins. in diameter at the butt and 11 ins. at the point. Each pile is cored in the center, the core being 4 ins. in diameter at the top and 2 ins. at the bottom. A water jet is operated through the core for sinking the pile. The sides of the pile coincide with the planes of an octagon, and are fluted at their middle by corrugations, the sections of each of which is a segment of a circle 2½ or 3 ins. in diameter. These corrugations form pas-

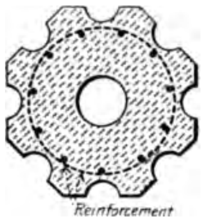


Fig. 260.—Gilbreth Corrugated Pile.

sages for the escape to the surface of the water forced through the core-hole in sinking the pile. They also increase the perimeter of the pile, giving a greater surface area for skin friction.

The piles are reinforced both transversely and longitudinally. Clinton Electrically Welded Fabric, with meshes 3 × 12 ins., was used for the pile here shown. The longer dimension is placed lengthwise with the pile, and is of No. 3 wire, while the horizontal transverse reinforcement is of No. 10 wire. No part of the reinforcement is closer than 1 in. from the outside of the concrete. For piles under ordinary conditions, only sufficient reinforcement is used to care for stresses due to handling. A greater area of metal should be used where severe conditions are met with, or where eccentric loading or bending will be brought upon the pile.

The piles are formed in moulds made of 2-in. plank. The corrugations are formed by nailing pieces on the inside of the form whose section is the segment of a circle. The sides of the form are fastened to end pieces, through which the core projects 6 or 8 in. During the progress of the moulding the central core is at intervals given a partial turn to prevent the setting of the cement holding it fast and prevent its removal. The forms are usually stripped from the piles from 24 to 48 hours after moulding, and thereafter the pile is kept moist to permit the proper action of setting to take place. This may be accomplished by covering the piles with burlap and sprinkling them from time to time. The piles should not be driven until they are at least 10 days old.

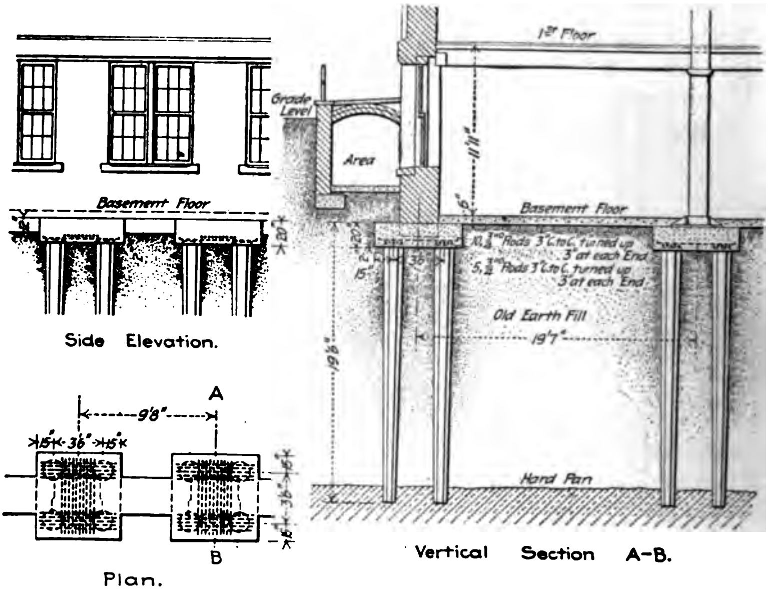


Fig. 261.—Gilbreth Pile Foundation for Lattemann Building, Brooklyn, N. Y.

The manner in which these piles were used for wall and column foundations for the Lattemann Building, Brooklyn, N. Y., is shown in Fig. 261.

Moulding Piles.—Piles may be moulded either vertically or horizontally, but much better results are obtained when they are moulded vertically. The shocks due to the action of the pile-driving hammer when driving are resisted by the concrete in a much better manner when the layers of the concrete are normal to the direction of application. Again, the principal stress carried by the pile is compression, and it is also best cared for when the layers of concrete are normal to the axis of the pile. Considerable difficulty is met with in moulding piles vertically, due to the confined space in which the concrete must be deposited and rammed. However, vertical piles may be constructed near each other and held in a rack or framework, thus reducing the area of the construction and storage yard.

Figures 262 and 263 show views of a plant where piles were cast, pile-driver and the process of pile-driving. Figure 263 shows piles and driver used for the new Union Station, Hamburg,

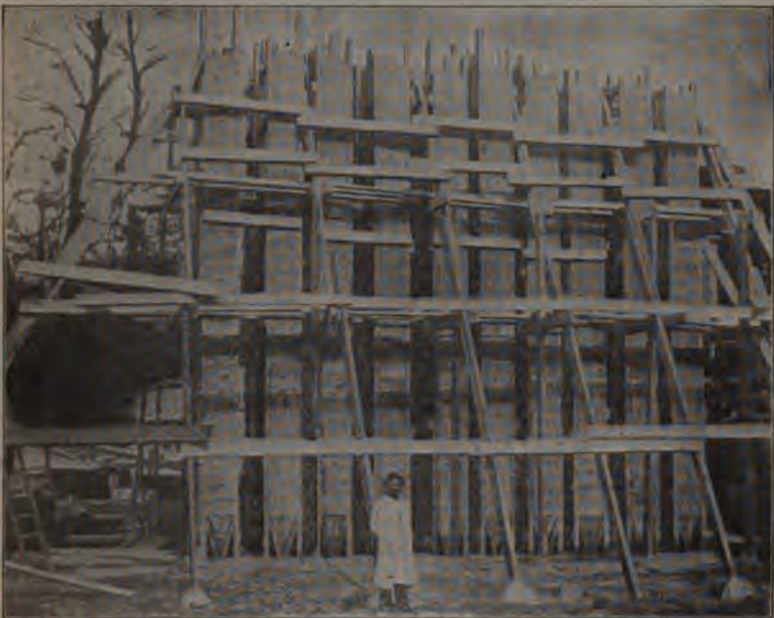


Fig. 262.—Forms for Molding Piles on End.

Germany. The piles used were 16 ins. sq., 45 ft. long, and carried 50 tons each. The weight used was 4 metric tons. The moulds used are similar to those used for columns, which are described in detail in another chapter. In moulding, the forms are placed at intervals, leaving sufficient room for working about the pile. Platforms are placed at different elevations, on which the men stand to place and tamp the concrete. In the Hennebique and similar piles the vertical reinforcing rods are bent together at the bottom of the pile, which is formed in an iron shoe similar to that used for timber piles. The cross-ties are usually slipped down from the top of the rods

Fig. 263.—Pile Driver and Pile for Union Station, Hamburg, Germany.

and placed at the required intervals apart when the concrete is deposited. When it is necessary to splice the rods they may be lapped from 1 to 2 ft., depending upon the size of the rods; or, better still, the rods may be dressed square on the ends and a butt joint made, with a sleeve of tubing over the joint. The tops of the rods are usually covered with 2 ins. or more of concrete. Sometimes the moulds are built in short sections and put up between vertical guide timbers as the concreting is being done. Another method is to build up all but one side, and add horizontal boards to this side as the concreting advances. Still a third method is to build the mould complete to the full height. When the latter method is used there is great difficulty in tamping the concrete, and it is necessary to resort to using a very wet concrete, with little ramming. The second method was used in the construction of the triangular pile described above.

M. Hennebique forms the hollow spaces on moulds of concrete in the construction of hollow piles; these moulds are left in place. These piles are as strong as solid piles and much lighter.

Horizontal moulding is more simple and economical, although the results obtained are not as satisfactory as in vertical moulding. The forms used and the methods employed are similar to those employed in moulding beams, and are fully explained in Chapter XXIV. This method may also be used in the construction of sheet piling, as in many cases the principal stress acts transversely to the axis of the pile.

The piles should set for a day after moulding without water; they should then be sprinkled every day for about a week, when they may be removed from the forms, and, if possible, set in a vertical position; or they may be laid flat on the ground or stacked with others until required for the work.

Piles should be allowed to stand about a month after moulding before driving, and as much longer as is possible.

Driving Reinforced Concrete Piles.—The methods of driving reinforced concrete piles are similar to those used in driving timber piles. The weight of the hammer is usually greatly increased and the height of the fall decreased. The weight of the hammer varies from 2,000 lbs. to 8,800 lbs., and the height of the drop should not exceed 10 or 12 ft. In driving 10 in. × 10 in. piles from 8 to 22 ft. long, for the foundation of the Dittman factory building, Cincinnati, Ohio, the weight of the hammer was 4,000 lbs., with an average drop of from 4 to 6 ft., and an average of about 90 blows to the pile were given. An attempt was made to use a greater drop; after a pile was started, two blows of 18 ft. and one of 16 ft. were given. The pile crushed under the 16-ft. blow. In driving the triangular piles shown by Fig. 257, steam hammers weighing 5,000 lbs., with a fall of 5 ft., were successfully used. In the construction of the wharf at Novorossisk, Russia, the piles of which are shown by Fig. 256, a 3,375-lb. hammer was used, with a drop of from 12 to 14 ft. The pile-driver used by Hennebique in constructing the foundations of the new Union Station at Hamburg, Germany, had a hammer weighing 8,000 lbs. Figure 263 is an illustration of this pile-driver, which was inclined at an angle of 10° from the vertical.

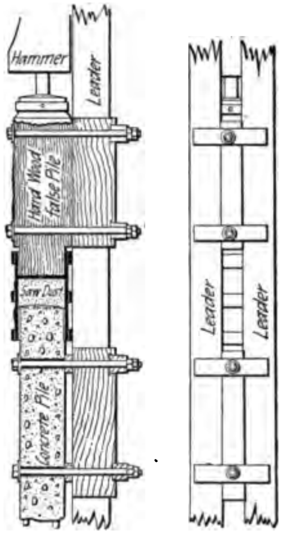


Fig. 264.—Driving Head for Work at Chantenay sur Loire, France.

Reinforced concrete piles are much heavier than timber piles, and on this account the pile-driver frames and tackle must have sufficient strength to lift the extra weight.

Special care is necessary to protect the head of a concrete pile during driving, otherwise it will be shattered by the shock of the hammer. Special devices have been designed to act as a cushion on the head of the pile. M. Hennebique employs a hollow steel cap filled with sand or sawdust, and calked below with clay and tow.

Sometimes he only uses a short timber false pile upon the head of the concrete pile. Another method employs a sheet steel tube or collar, reinforced with steel rings. The diameter of this tube is slightly larger than the head of the pile, and secured to it with hardwood wedges; it projects two-thirds of its length above the head of the pile. Alternate layers of shavings and sawdust are placed in the tube, which, when compressed, occupy about half the remaining space to the top. The purpose of the shavings is to reduce lateral thrust of the sawdust. A hardwood false pile, loosely fastened to the guides, is fitted into this collar and receives the blows of the hammer. This device is shown in Fig. 264, and was used in driving sheet piling at Chantenay-sur-Loire, France. A sand cushion did not prove successful in driving the piles for the Dittman factory, mentioned on page 455. The cap was stuffed with pieces of old rubber hose and a wooden false pile placed upon the metal cap to receive the blows of the hammer.

In many situations a water jet must be used for sinking piles; in others it can be used in connection with the pile-driver. When it is the intention to sink piles with a water jet, the pile is cast hollow or has a jet pipe cast in the concrete. The jet is applied at or as near as possible to the point of the pile. It loosens the material around the point of the pile, renders it more fluid, and the pile, on account of its weight, sinks into the loosened material. Sometimes weight is applied at the top of the pile, or it is lightly rammed with a pile-driver. This process continues until the pipe reaches the depth desired. The water jet is most efficient in clear sand, quicksand, mud or soft clay. In gravel, or sand combined with gravel, or hard clay, the water jet is almost useless.

The operation of driving the piles for the Brooklyn foundation, shown by Fig. 261, which consisted of 480 piles driven through from 15 to 20 ft. of earth fill to hardpan, was as follows: A pile-driver such as is used for driving ordinary wood piles, was employed, but with a much heavier hammer and less drop. The jetting is accomplished by inserting a 2-in. pipe within the pile. The pipe was tapered at the bottom to a diameter of 1 in., forming a nozzle. A water pressure of about 120 lbs. per sq. in. was used.

A special cap, shown in Fig. 265, was used. This cap is about 3 ft. in height, and the bottom end fits over the head of the pile. In one side of this cap is a slot from the outside to the center. This allows an opening for the projection of the jet water pipe. The outside of the cap is formed of a steel shell, while the inside has a compartment filled with rubber packing, and the top has a wooden block which receives the blow from the hammer. In this way the head of the pile is cushioned, thereby preventing the injury of the head of the pile during driving.

During the operation of driving, the water from the jet comes up on the outside of the pile and carries with it the material which it displaces during driving. This, with the blows from the hammer, forces the pile into place. It is said that the action of the water puddles the earth firmly about the pile and increases the

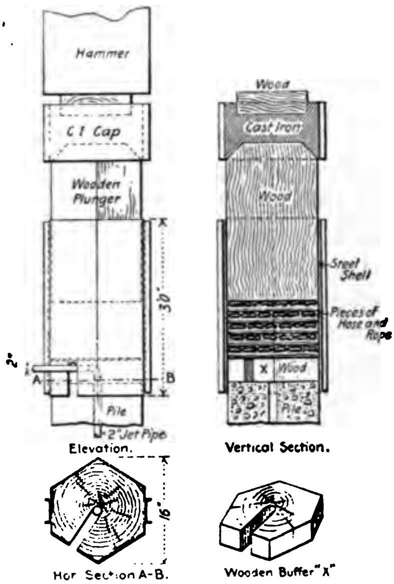


Fig. 265.—Driving Head for Gilbreth Piles.

skin friction beyond what usually exists when the pile is driven by a hammer alone.

Armored Timber Piles.—Reinforced concrete has been successfully used for armoring timber piles in teredo infested waters. Christophe, in "Beton Armé," page 326, gives an example of this kind of construction. A bridge over Cockle Creek, near Sidney, New South Wales, has five three-pile bents armored with Monier cylinders. The soil through which the piles were driven consisted of a mixture of sand and vegetable matter about 5 ft. thick, overlying a stiff blue clay. Iron bark piles, 18 ins. in diameter at the butt and 14 ins. at the point, and 40 ft. long, were driven about 15 ft. into the clay stratum. Hardwood battens were spiked along the length of the pile and spaced 90° apart on its circumference. These strips extended from the level of the clay stratum up to high water. Monier pipes having a diameter of 21 ins. were placed about the pile. These pipes were made of cement mortar having a thickness of 1⅜ in., and reinforced with a 1¼-in. square mesh netting of No. 16 gauge wire. The joints between adjacent sections were spliced with a sleeve of Monier netting, covered with cement mortar.

A sufficient number of sections were joined together to make a

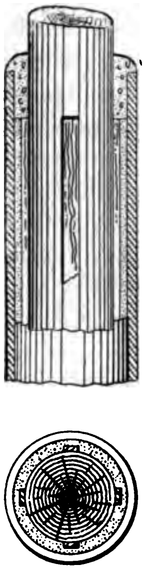


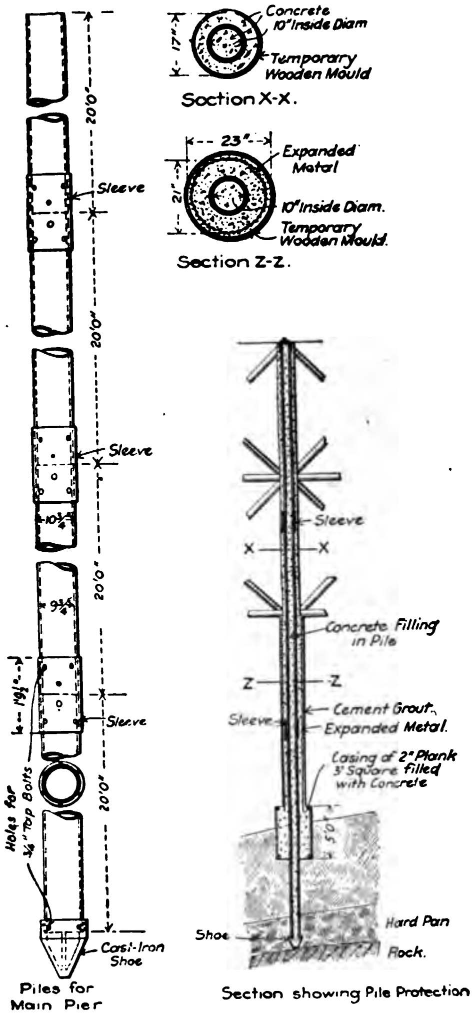
Fig. 266.—Armoring for Timber Piles, Cockle Creek Bridge, New South Wales.

tube long enough to extend from high water level down to the clay stratum. After the tube was completed it was lowered until it rested on the bottom of the stream. It was then sunk until it rested upon the clay stratum by means of a water jet, and pressure applied at the top. The battens on the sides of the piles acted as guides to keep the casing and pile concentric. The annular space between the piles and the shell was scoured out by means of the water jet and filled with clean sand, with a 9-in. cap of cement mortar at the top. Figure 266 shows details used in this construction.

Armored Steel Piles.—Reinforced concrete was used as an armoring for metal piles in the construction of a pier for the United States Coaling Station at Frenchman's Bay, Maine. The main pier has four-pile transverse bents, 48 ft. long and 10 ft. apart longitudinally. The piles are steel tubes ½ in. thick and 10 ins. inside diameter. They are made of 20-ft. sections spliced by outside sleeves and braced in both transverse and longitudinal directions by horizontal and diagonal angle struts, which extend about 10 ft. below low water, and riveted to collar plates bolted to the piles. The piles were shod with conical cast-iron points, bored to fit the piles, and furnish full bearing for the ends of the pipes, to which they were tap bolted. The ends of the pipes were faced and turned to a driving fit in the sleeves by which they were spliced. The latter were also of ½-in. metal and fastened to the lower sections of the pipe with counter-sunk shop-driven rivets. Field connections at the top ends of the sleeves were made with tap bolts. The piles were driven to solid rock through 12 ft. of sand, gravel and clay and 2 ft. of hardpan. The driving was done by a 2,000-lb. hammer, assisted by a water jet fastened to the pile point and withdrawn after the pile was driven. Figure 267, with cross-sections X—X and Z—Z, show the character of the reinforcement. Figure 268 is a longitudinal section of a pile in place.

The method of placing the reinforcement is as follows: After each pile was driven a diver made an excavation around it, in which was set a wooden casing 3 ft. square and 5 ft. deep. This was sunk to its full depth below the bottom of the bay, or to hard bottom at a less depth. This casing was filled with 1 : 3 : 6 Portland cement concrete, in the upper part of which was seated the lower end of a concrete reinforcing tube 22 ins. in diameter of No. 10 expanded metal with a 3-in. mesh. This meshing extended up to the lower horizontal struts. A wooden sheathing 1 in. thick and 24 ins. in diameter was placed about the whole and filled with 1 : 2 Portland cement mortar. A protecting coating 17 ins. in diameter of 1 : 2 Portland cement mortar extended from the level of the lower strut to the pile cap. In this portion, however, the metal reinforcement was omitted. Experienced divers were employed to do the work, and great care taken to make the joints between sections of the casing watertight. The wooden sheathing was removed after the concrete had set. The interior of the was filled with 1 : 3 : 5 Portland cement concrete, made of n. stone and tamped with a heavy iron rammer.

aisson Foundations.—A form of construction which has been



267-268.—Armoring for Steel Piles, U. S. Coaling Station, Frenchman's Bay, Maine.

nsively used for bridge piers consists of a thin metal cylinder c into the soil to any desired depth and filled with concrete. imes this cylinder is placed about a cluster of piles. In many situations a reinforced concrete cylinder may be substituted for this metal shell and afterwards filled with concrete.

The following, taken from Beton Armé, is a good example of this form of construction: Two Monier cylindrical piers were used in the construction of the Cockle Creek Bridge, New South Wales, Mr. E. M. De Burgh, M. Inst. C. E., Engineer. The cylinders were 3 ft. 6 ins. internal diameter. The shells were 2⅛ ins. thick and had a length of 3 ft. 7 ins. The reinforcement con-

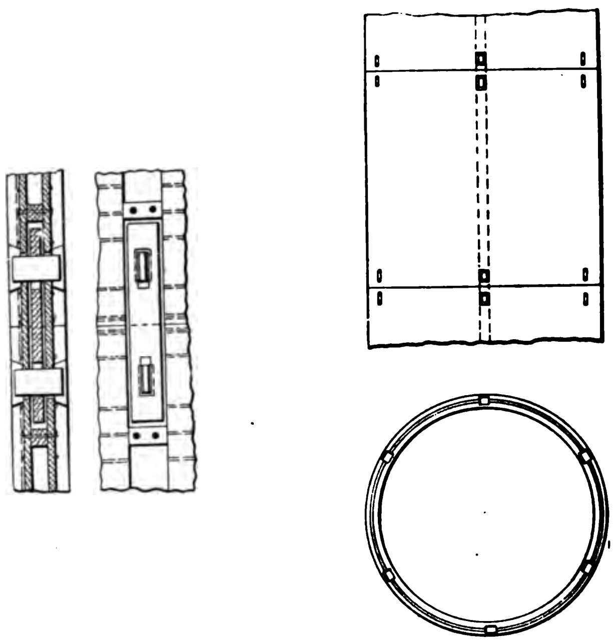


Fig. 269.—Reinforced Concrete Cylinder Foundation, Cockle Creek Bridge, New South Wales.

sists of No. 16 gauge wire woven to 1¼ in. mesh, with two spirals of No. 8 gauge wire wound about the cylinder, with a pitch of 1 in. between adjacent coils. The splice was formed by small vertical splice bars, which are placed between six pairs of bars 1¾ × ¼ in., inserted between the spiral coils and extend from end to end of the cylinder, and equally spaced about the circumference. Mortices were cut in the longitudinal bars and splice bars and steel keys inserted, locking the whole together. Figure 269 shows the construction of the cylinders and locking device.

A cast-iron cutting edge was fitted to the bottom cylinder and the cylinders sunk by excavating from within, successive lengths being added as is done in sinking cast-iron cylinders. These tubular piers were sunk through the silt, sand and gravel to a depth of 36 ft. into hard blue clay. The sinking of these piles was accomplished under ordinary air pressure, but there is no reason why pneumatic caissons of a similar type can not be successfully driven. When the tubes were firmly seated in the clay they were filled with concrete. The cost of the tubes, delivered, was $6.00 per lin. ft.

A modification of this form of construction was adopted by the

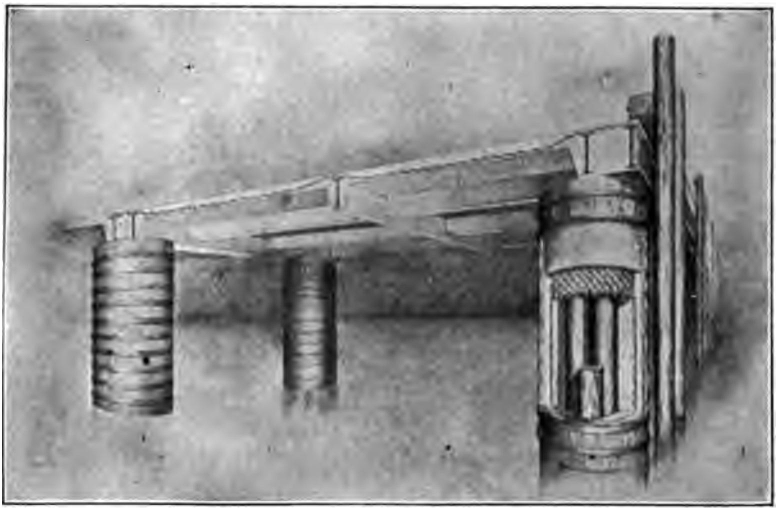


Fig. 270.—Armoring for Timber Piles, Union St. Wharf, San Francisco, Cal.

State Harbor Commission, San Francisco, Cal., for pier construction in toredo infested waters, and used in the construction of the Union Street Wharf, San Francisco. Figure 270 shows the details of the caissons and the wharf floor supported by them. Three carrying piles were used. These were cut off, one 2 ft., one 6 ft., and one 8 ft. below the wharf level. Wooden stave cylinders of 3-in. plank were placed about the piles and driven firmly into the mud and sealed at the bottom. They were then pumped out; a cylinder of No. 16 gauge expanded metal having a diameter 6 ins. less than the surrounding cylinder was then put in place and the tube filled with concrete.

Reinforced concrete may be used to replace the usual metal or wood framing used in the construction of pneumatic caissons for pier foundations and other subaqueous works. Recently this method has been employed in the construction of the large tunnel caissons on the Jersey shore connecting the tunnels of the Hudson Co. crossing the Hudson River. The first caisson constructed is approximately trapezoidal in plan. It is about 101 ft. long and from 23½ to 46 ft. wide, with an extreme height above the cutting edge of 51½ ft. The total weight of the permanent structure was about 6,650 tons. A detailed description of the caisson is given in Engineering Record for September 29 and October 6, 1906. The American Concrete Caisson Co., of Troy, N. Y., is developing several types of caissons, constructing the walls, bracing, top and cutting edge of reinforced concrete. Among the advantages claimed for such caissons are: rapidity of construction, avoiding slow deliveries and time necessary for fabrication of steel work; lower cost and additional weight obtained by use of concrete, thereby eliminating heavy ballasting necessary when a wooden or metal caisson is used.

CHAPTER XXIII.

GENERAL BUILDING CONSTRUCTION.

The fire resisting qualities of Portland cement concrete makes it especially desirable as a building material; this, with its property of preserving the metal from corrosion, makes structures in which it is used almost indestructible. Reinforced concrete may be used equally well for foundations, floors, columns, girders, walls, roofs and stairways, making all parts equally fireproof. The actual cost of reinforced concrete in office and warehouse building construction, as compared to steel, is from 10 to 20 per cent. less than the latter when protected by the usual fireproofing material. This saving in the construction material will often afford a saving of from 3 to 5 per cent. on the total cost of the building, and means the saving of quite a substantial sum in a building the size of the Ingalls Building.

By using flat floor slabs and locating girders and beams at partition and wall lines, a great saving in the thickness of floors over the ordinary steel beam and girder construction is secured. This item reduces considerably the height of a high building, while retaining the same height of ceilings.

When used in the construction of factory buildings, reinforced concrete will be found to occupy an intermediate position between steel construction with fireproofing and the usual type of slow burning construction, consisting of brick walls, timber columns and floor beams, with wood floors. The cost of the reinforced concrete building will often run up to 10 or 12 per cent. in excess of the latter type of construction. The first cost of ordinary factory buildings of two or three stories in height and say 50 x 250 ft. in plan, containing no unusual features, when constructed of reinforced concrete, as compared to ordinary slow burning construction, depends upon the locality. In New York and New England the first cost will be found to average from 2½ to 7 per cent. in excess for the reinforced concrete building. In the Southern States it may be as much as 25 or 30 per cent. in excess of the wood construction, depending upon the relative cost of cement and timber. However, the lowest first cost may not be the cheapest construction, as insurance rates on the reinforced concrete buildings are much lower than on the slow burning wood construction. In Brooklyn, N. Y., a rate of 20 cts. was obtained for the R. C. Hanan & Son's reinforced concrete shoe factory, while the rate on an adjacent building of slow burning wood construction was 45 cts. per $100. The vibration due to rapid running machinery is much less in the reinforced concrete building, hence there will be much less jar and correspondingly less trouble in keeping light machinery in running order. The life of the machinery will, therefore, be prolonged, and cost of maintenance much reduced. When these elements are considered in connection with first cost it will be found in many if not all cases that the balance is much in favor of the reinforced concrete building.

Building Construction.—Reinforced concrete may be used as a structural material to replace masonry, metal or wood in the construction of buildings. Buildings have been constructed entirely of this material, from the foundation to the roof. A notable example of such a construction is that of the 16-story Ingalls Office Building, Cincinnati, Ohio. This building is 100 × 50½ ft. in plan, and 213 ft. high from the sidewalk to the cornice. The material used is entirely reinforced concrete, except the marble and brick veneering for the walls, the metal window frames and inside trimmings.

Reinforced concrete has been much more extensively employed in the construction of buildings in Europe than in America. As a building material it has been largely employed in this country for some years in the construction of foundations and fireproof floors, but within the past two or three years it has come into greater favor as a general structural material and is being extensively employed in the construction of factory buildings, store houses, power houses, and its use is being gradually extended to all classes of buildings. Reinforced concrete may be used equally well in the construction of floors, columns, walls, partitions, roofs, stairways and foundations.

Columns.—Columns used in building construction are generally of two kinds, viz.: Those of rectangular or polygonal section, reinforced with straight rods tied together at intervals, and hooped columns. In the first form a reinforcing rod is usually placed at each corner or angle only, but at times an additional rod is placed t the middle of each side. In the hooped column the function f the hooping is to increase the compressive strength of the con-

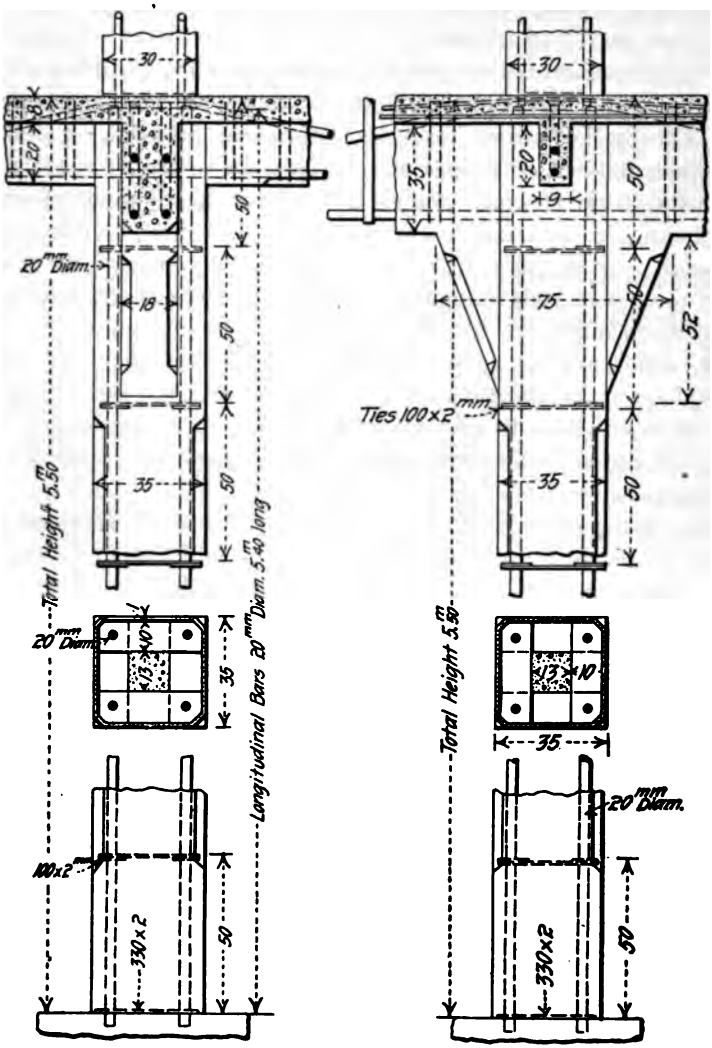


Fig. 271.—Hennebique Column.

ete, which alone takes all compressive stress, while slender, raight rods are added to take care of possible bending stresses.

Hennebique Columns.—An early type of Hennebique column is iown in Fig. 271. This column is described by Christophe in

"Beton Armé," and was used in the construction of the Palais de Justice at Viviers, France. This column is 35 cm., about 14 ins. square and 5.5 m. (18 ft.) in length, and was designed to carry 43 metric tons (47.4 short tons).

The usual features of Hennebique columns are as follows: A reinforcement consisting of four or more vertical rods, tied together in the earlier form by connecting flats having holes punched in them of proper size to pass the rods. These flats, however, form cleavage planes in the concrete, thereby greatly weakening it. In a later form of column M. Hennebique ties the vertical rods together with strap iron placed on edge or with wire ties. The rods employed vary usually from about 5-16 ins. to 2 ins. in diameter. At the base of the column the ends of the rods, which are cut square, rest on a horizontal plate of metal embedded in the concrete. It is usual to make the reinforcement run from one floor to another, and when joints are necessary the ends of the rods are cut square and enclosed in a cylindrical sleeve.

Fig. 319, page 498, shows a good example of a Hennebique column used in the construction of a store building in Chicago. The reinforcing rods vary from 2 $^{9}/_{16}$ ins. at the first floor to 1 in. in the seventh and top floor. The rods are tied together at intervals of about 2 ft. with ⅜-in. square bands. These bands have loops of wire, twisted tightly, connecting the centers of the opposite sides and offering additional resistance against the tendency of the vertical rods to spread. At the bottom of the column the rods rest on a 24 × 24 × ½-in. plate bedded in the concrete footing. The joints occur 6 ins. above each floor level, and are made with a sleeve of gas pipe, into which the ends of the two sections of a rod are wedged. There are a number of column systems, so-called, belonging to this type, which differ from the Hennebique column only in the form of tie used to bind together the vertical reinforcing rods. A column of this type designed to carry eccentric loading is used in the screen house of the Ontario Power Company, of Niagara Falls, Ontario. These columns are 12 × 15 ins. in section and support the direct load of heavy roof beams and an eccentric load from a 60 × 20-in. reinforced concrete crane girder resting upon brackets upon the inner sides of the columns. The column reinforcements consist of four rods, 1¼ ins. in diameter, fastened together by ties in the usual manner. The bracket extends 15 ins. from the column to which it is tied and is 12 ins. wide. The way in which the steel members are tied together is shown in Fig. 272. The lateral tie rods used to join the bracket to the column are 1 in. in diameter. The method of reinforcing the roof and crane girders is also shown in Fig. 272. A 1:3:5 concrete was used for this building, the stone being crushed to pass a ¾-in. screen.

A modification of the usual Hennebique type of column was used in the 16-story Ingalls Office Building, Cincinnati, Ohio.

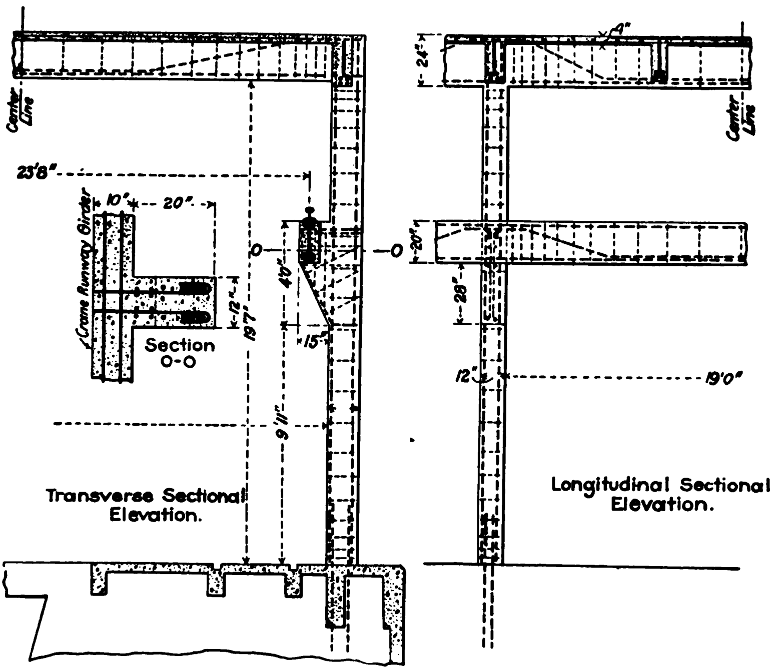


Fig. 272.—Column and Crane Girder, Ontario Power Co.'s Power House.

This column is shown in Fig. 273. These columns are designed to resist lateral pressure due to wind. In the Ingalls Building the columns are spaced 16 to 33 ft. centers. The larger columns decrease in size from 34 x 38 ins. at the bottom to 12 x 12 ins. at the top. The concrete footings were built independently of the columns, and are of sufficient size to properly distribute the column loads to the subsoil. Upon the footing was placed a cast-iron base plate having circular projections on top to form seats for the vertical compression rods. The tops of the projections were faced to a true horizontal plane to form a true bearing for the rods, which had their ends also faced. Each column had 4, 6 or 8 plain round reinforcing rods from 2 to 3½ ins. in diameter. All joints were carefully made, with the ends of the rods faced. In the lower part of the building these rods extend through one story only, but above the third story they extend the height of two stories. At each splice a sleeve is put around the top of the bar to form a socket for the next bar above. When ready to place the upper bars, they are set in the sleeves, grouted, and the concrete carried above the joint. These large circular bars are to take compression only, and in addition to them each column has from 4 to 10 smaller bars of twisted steel to take the tension due to wind loads. The series of upright rods in each column are surrounded by rectangular hoops of twisted steel,

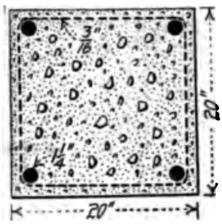


Fig. 273.—Column for Ingalls Building.

spaced at vertical intervals of about 12 ins., the end of each hoop being lapped and bound with wire, while the hoops are secured by wire ties to each of the upright bars with which they come in contact. The function of these steel hoops is primarily to resist the tendency of the compression bars to buckle, but they also greatly increase the shearing strength of the columns. The column loads coming upon the shoes range from 300 to 700 tons.

Wall columns are usually of rectangular section, although tee sections are also used. Sometimes, when the loads are light and stiffness with a pilaster effect is desired at the column line, a large hollow section is used. The reinforcing almost always employed consists of straight rods tied together with occasional horizontal ties or bands. Fig. 274 shows cross-sections of hollow side and corner columns used in the construction of the Kelly & Jones factory building, Greensburg, Pa.

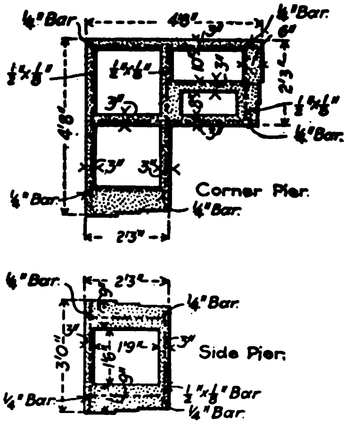


Fig. 274.—Columns, Kelly & Jones' Factory.

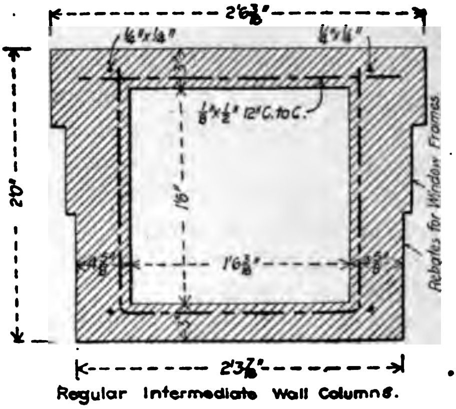


Fig. 275.—Column, Pacific Borax Co.'s Factory.

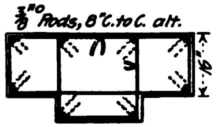


Fig. 276.—Wall Column, Twelve-Story Loft Building.

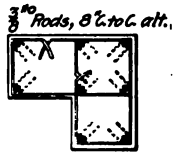


Fig. 277.—Corner Column, Twelve-Story Loft Building.

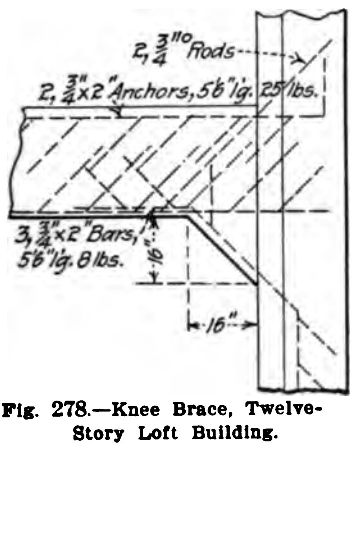


Fig. 278.—Knee Brace, Twelve-Story Loft Building.

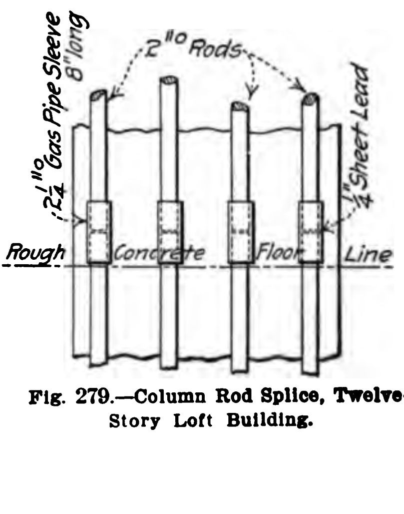


Fig. 279.—Column Rod Splice, Twelve-Story Loft Building.

Fig. 275 shows another example of a hollow intermediate wall column used in the Pacific Coast Borax Factory Building, Bayonne, N. J.

Fig. 276 shows the section of a rectangular wall column, and Fig. 277 that of a corner column being used in the construction of a 12-story loft building in New York City. Fig. 278 shows the detail of a knee-brace used on all wall columns. Fig. 279 shows a detail of column rod splice. Fig. 280 gives detail of a column footing so designed as to distribute the pressure on the reinforcing rods, while Fig. 281 gives an enlarged detail of one of the cast-iron shoes.

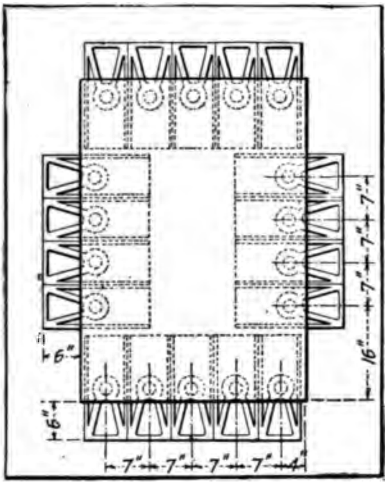


Fig. 280.—Column Footing, Twelve-Story Loft Building.

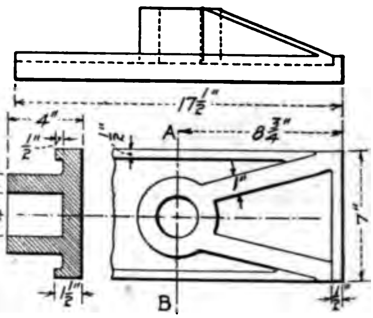


Fig. 281.—Cast-Iron Shoe for Concrete Column, Twelve-Story Loft Building.

Fig. 282 shows typical column sections used in the factory of the American Oak Leather Company, Cincinnati, Ohio. The sectional area of metal used for reinforcement was from 1 to 2 per cent.

Hooped Columns.—The columns used in the Kelly & Jones Co.'s reinforced concrete building were among the first hooped columns used in this country. These columns are square in section at top and bottom, with the corners chamfered off to give them an octagonal section throughout the body of the column. Their sizes were 23, 20, 15 and 10 ins. in the first, second, third and fourth floors, respectively. The reinforcement consists of eight ¼-in. vertical rods spaced at equal intervals in a circle about ½ in. inside of the surface of the concrete. About these rods was placed the hooping, consisting of a helix of 4 ins. pitch, made of ¼-in. twisted steel, extending from end to end of the column. The helix and vertical rods were wired together at intersections. Fig. 283 shows section of this column. The columns used in an addition to the Pacific Coast Borax Fac-

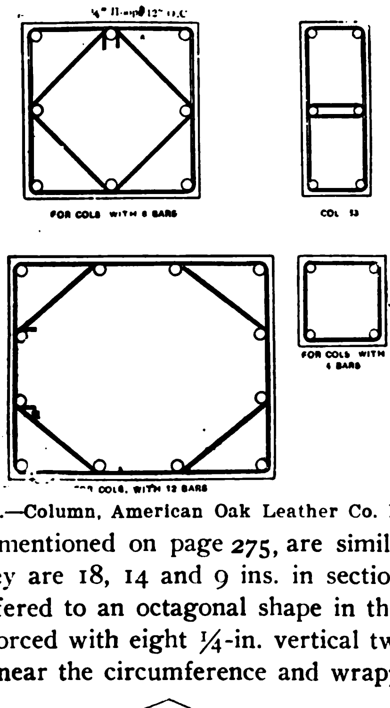


Fig. 282.—Column, American Oak Leather Co. Factory.

tory Building, mentioned on page 275, are similar to those just described. They are 18, 14 and 9 ins. in section, square at the ends and chamfered to an octagonal shape in the middle. Each column is reinforced with eight ¼-in. vertical twisted steel rods, equally spaced near the circumference and wrapped with a helix

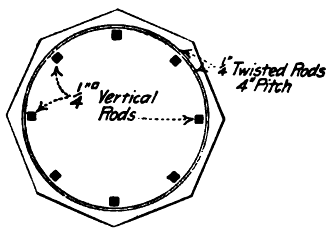


Fig. 283.—Hooped Column, Kelly & Jones Factory.

of ½ × ⅛-in. steel rod with a 4-in pitch (Fig. 284). A different arrangement of the reinforcement was used in the columns for the shops of the United Shoe Machinery Co., Beverly, Mass. Columns of octagonal section, with diameters of 22, 18¼, 14 and 8 ins. in the first, second, third and fourth floors, respectively, were reinforced with eight vertical reinforcing bars, ¼ in. square, of twisted steel rods, in the angles close to the surface and with an interior coil of ¼-in. twisted steel hooping, with 4-in. pitch, inserted within the vertical rods. The vertical rods were of one-story lengths, having 12-in. splices wrapped with a coil of ⅛ x ¼-in. steel. A cross-section of this column is shown in Fig. 285. As has been stated in Chapter XXI., the experiments of Considère, supported by tests of other experimenters, show that the best results are obtained when the pitch of the hooping is from 1-7 to 1-10 the diameter of the columns. In the above columns the distance between adjacent coils is in all cases considerably more than this. The amount of metal used is less than is necessary to secure to the fullest extent the benefits of hooping. The above columns show conservative designing, which is commendable, inasmuch as this form of reinforcing is comparatively new and

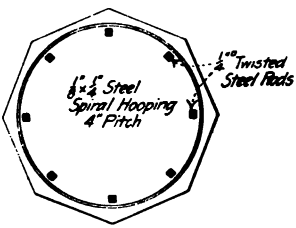


Fig. 284.—Hooped Column, Pacific Coast Borax Co.'s Factory.

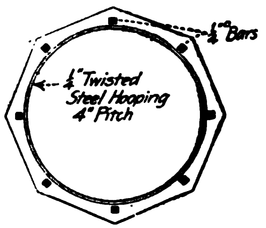


Fig. 285.—Hooped Column, United Shoe Machinery Co.'s Factory.

untried. The column section shown in Fig. 285, as will be noted, has the longitudinal rods outside the hooping. Should the column become overloaded to such an extent as to cause the concrete outside the hooping to shell off the resisting power of these rods against flexure will be lost at the time when most needed. Again, when in this position the rods cannot assist the hooping in restraining the concrete from flowing laterally.

The Cummings Column.—In place of the spiral steel hooping used in the Considère column, hoops are used by Mr. Robert A. Cummings for column reinforcement. The hoops are made of flat steel bent to a circle, with the ends of each hoop riveted together. One end of the steel is bent outward at right angles to the hoops and projects a uniform distance from the hoops to hold the reinforcement concentrically with the column in a fixed position in the mould. Vertical reinforcements are also used, and consist usually of angles with small holes punched at intervals for staples, by means of which the hoops are easily secured to them. The hoops are spaced at regular intervals of from 2 to 3 ins. The arrangement makes a rigid reinforcement which is easily assembled and placed in the mould in its permanent position (Fig. 286).

Expanded Metal Hooping.—A novel application of the hooping principle is shown in the columns recently used in the construc-

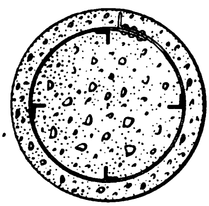


Fig. 286.—Cummings Column.

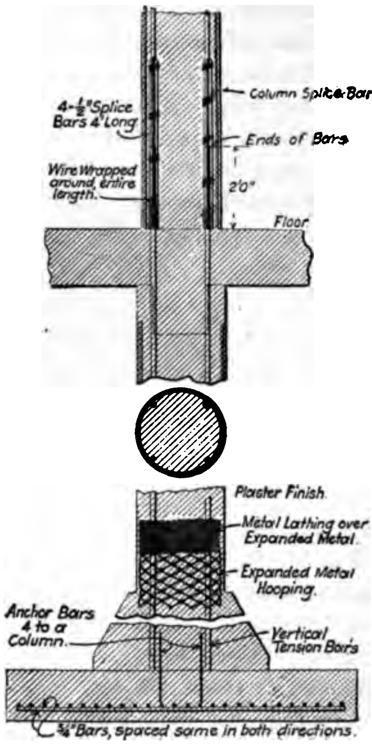


Fig. 287.—Column, Thompson & Norris Factory.

tion of the Thompson & Norris Factory Building, in Brooklyn, N. Y. A section and elevation of this column is shown in Fig. 287. The hooping consists of No. 10 expanded metal, with a 3-in. mesh. The sheets of metal were accurately curved to cylindrical shape, with the edges overlapped and wired so securely that the bond has sufficient strength to break the body of the metal. This metal was wrapped with No. 24 expanded metal lath with a ½-in. mesh. The metal lath retained the concrete when deposited, thereby enabling the columns to be constructed without any other form of mould. Provision was made for bending stresses by four vertical corrugated rods placed inside the curved hooping cylinder and wired to it at equal distances apart. These vertical rods varied in size from 1 in. in the basement to ½ in. in the sixth story, and are spliced 2 ft. above each floor, the rods being butt-jointed and spliced with four ½-in. fish-bars 4 ft. long securely wrapped to them with 3-16-in. wire. These columns vary in size from 28 ins. in diameter at the basement to 12 ins. in the upper stories, the diameter being exclusive of the plaster finish, which was ⅝ in. thick. The columns are of uniform diameter throughout the height of each story. The concreting was done in the usual manner and finished with a ⅝-in. face coat of 1 : 3 cement mortar troweled smooth. The hooping was designed to sustain a bursting pressure of $\frac{1}{4.8}$ of the compression due to vertical

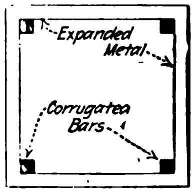


Fig. 288.—Column for Warehouse, Toronto, Canada.

load. The latter stress was 750 lbs, giving a tension of 156 lbs. to be provided for by the hooping.

An examination of these columns showed that the expanded metal lath did not in all cases fulfill the function for which it was intended. When a wet mixture was used the cement near the perimeter of the column drained out with the excess of water, leaving only the coarse particles adjacent to the metal, thereby destroying the full bearing against the metal which is essential to secure the full strength of a hooped column. A tolerably dry mixture is necessary for columns of this character. To place a dry mixture much care is necessary, and when properly done the saving by the use of expanded metal in place of the usual column mould will not be very great.

Fig. 288 shows a section of a column used in a warehouse building in Toronto, Canada. The reinforcement consists of four vertical rods, surrounded on all four sides of the column with expanded metal. When expanded metal is thus placed it is evident that it will not, in a strict sense, act as hooping, but probably some additional strength will be secured. It has, however, another and important function, as it acts as a form for the concrete; and if a moderately dry mixture be used and the mesh of the metal be not too large, no other kind of mould will be necessary, thereby reducing considerably the cost of construction.

Hooped Cinder Concrete Shells.—In the construction of a six-story factory for the Bush Terminal Company, Brooklyn, N. Y., hooped columns of a unique design were used. An outside shell of cinder concrete was used to hold the hooping in place, to act as a form for the concrete core and to protect the metal against fire.

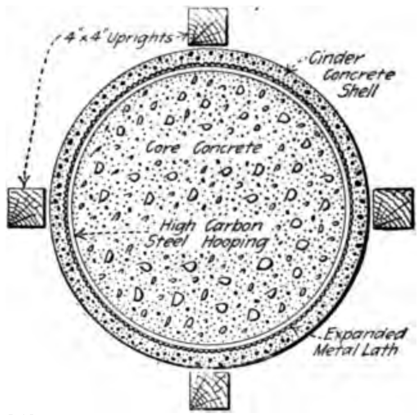


Fig. 289.—Column for Bush Terminal Co. Factory.

The shell, which has a thickness of about 1½ ins., was constructed in sections about 2 ft. long. The reinforcement, which consisted of high carbon steel wires, from 3-16 to 5-16 in. in diameter, with an elastic limit of about 55,000 lbs. per sq. in., was wrapped about a collapsible mandril with a pitch of about 2 to 3½ ins. At either end a partial extra turn of the rod was made and wrapped to the outer coil by means of soft wire. A sheet of expanded metal lath was then wrapped about the coil to act as an inner form for the shell. The expanded metal was spliced with a lap of about 4 ins. and the spiral rod and expanded metal wired together where necessary. The metal was then removed from the reel and placed within a mould about 4 ins. greater in diameter than that of the hooping. The annular space between the expanded metal and the mould was then filled with cinder concrete. When the concrete had hardened the shell was removed from the form.

In the construction of the column uprights formed of 4 × 4-in. timbers (Fig. 289) were set up at the four sides of the column to support the girder and beam forms framing into it and to keep the rings in place. The shells were put in one after another and the core concrete put in place as one shell was placed upon another. Each shell acted as a unit and no splicing was found necessary between adjacent shells. The core concrete bears directly against the hooping, which supports it against any tendency to spread laterally when subjected to stress. A working stress of 1,000 lbs. per sq. in. was allowed upon these hooped columns. A test load of double this amount was brought upon a number of columns with no signs of failure. The main interior columns for this building have a diameter of 33 ins. in the basement, which decreases in successive upper stories to 30½, 29, 26½, 23½, 19 and 12 ins. over all.

Fig. 290 shows detail of hooped column recommended by the Monolith Steel Co., of Washington, D. C. As will be seen, the monolith bar is used for vertical reinforcement. The method of attaching the hooping to the vertical rods should be noted.

The Use of Metal Columns in Reinforced Concrete Structures.— The comparatively low stresses which are used for reinforced concrete columns necessitate a column section of considerable size when used for heavy loads in buildings several stories in height. Economy of space dictates the use of columns of small section. This necessitates the use of metal columns for interior columns. The slow deliveries of structural steel make the use of built-up steel columns in many cases impracticable. This has led to the use of cast-iron columns. It is unfortunate that steel cannot be obtained for several months after being ordered, as built-up steel columns can be used in many situations with economy, and if proper care be used in working up details very satisfactory connections with reinforced concrete girders may be secured.

Considerable difficulty has been experienced in securing proper connections of girders and beams to cast-iron columns. It is necessary in almost all cases to secure a rigid connection of girders and beams to columns. The author has seen concrete girders resting on seats cast on the side of the cast-iron columns with no other connection than one or two plain rods hooked through holes in lugs also cast on the side of the column. These connections were used in a building eight stories in height subject to more or less vibration from machinery on the floors. Such connections cannot be too severely criticised. It is possible to secure satisfactory connections if sufficient care is taken in working up details. A detail which has been used in several loft buildings consists of cutting away the metal at the middle of the sides of the column and increasing it at the corners, leaving open-

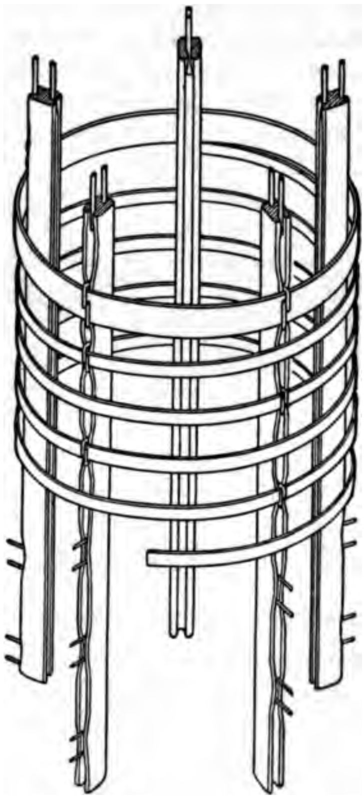


Fig. 290.—Hooped Column, Monolith Steel Co.

ings through which the girders are run. Suitable seats must be provided and the section increased considerably at the corners to secure sufficient area of metal to transmit the stresses to the main body of the column below. This necessitates a considerable increase in the thickness of metal at these points, causing dangerous shrinkage strains. This and the unsatisfactory workmanship usually obtained when any unusual features are introduced on cats-iron work makes this detail more or less undesirable.

Another detail consists of providing suitable seats for carrying the girders and beams and anchoring them to the column and joining them together by running continuity bars through holes cored or drilled in the columns. The comparatively small holes necessary for the rods do not weaken the column section, and as stiff a connection is obtained as if the girders were run through the columns. Fig. 291 shows details of the connections of girders to column being used in the construction of a twelve-story loft building in New York City. This structure was designed by the Trussed Concrete Steel Co., No. 1 Madison Avenue, New York

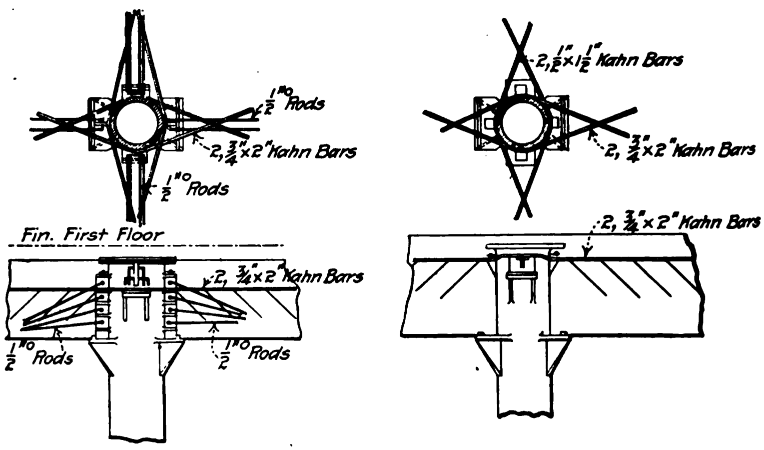


Fig. 291.—Column and Girder Construction, Twelve-Story Loft Building.

City. The Kahn bar was used for reinforcement in this building. A very rigid connection is secured by running continuity bars around the columns and extending them several feet into the girders on either side. Fig. 292 shows a detail used when it is necessary to support a reinforced column in the upper stories upon cast-iron columns below. In many cases such construction is desirable, as only comparatively small concrete columns are necessary in the upper stories.

Beam, Girder and Slab Construction.—When used in floors, reinforced concrete constructions may be divided into two general classes, viz.: when used as a filling between the beams and girders of a framed steel floor system, and when the construction is entirely of reinforced concrete in the form of a slab with or without reinforced concrete girders or arches, the whole, in either event, being built as a monolith. A modification of the second class consists of a monolithic construction placed between the main steel girders or beams, the slab, strengthened where necessary by reinforced concrete ribs, replacing the small beams and concrete filling slab of the first class.

Filling slabs may have either a flat or arched form. The flat slab, forming the floor plate, may have any of the following forms: (1) It may rest directly upon the top flange of the supporting beam; (2) it may be supported by the bottom flange of the floor beams; (3) it may be flush with or embed the top flange. The slab in the latter form is sometimes supported by a corbel filling of concrete resting on the bottom flange.

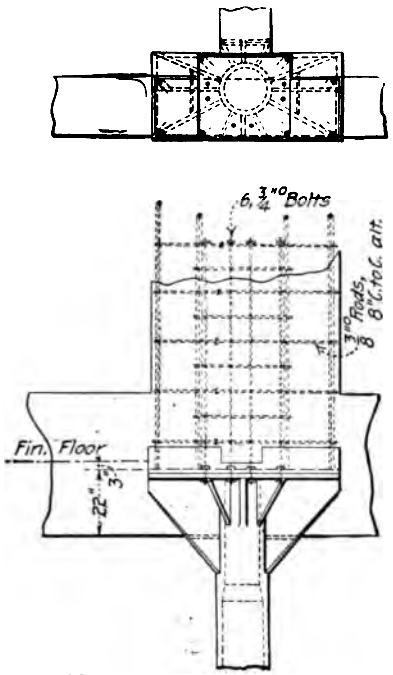


Fig. 292.—Method of Supporting Concrete Column on a Cast-Iron Column.

Arched construction may consist (1) of an arch ring sprung from the bottom flange of the beams with some form of filling between the extrados and the level of the floor surface; (2) of a flat-topped arch sprung from the bottom flange of the beams, having its flat extrados flush with or above the top of the beams.

Monolithic floors may consist of (1) flat floor slabs of uniform thickness; (2) flat floor slabs, strengthened by reinforced concrete beams, sometimes called ribbed slabs; (3) arches without ribs, and (4) arches with ribs. The usual type of monolithic ribbed floor consists of heavy reinforced concrete ribs or girders supported by walls and columns and arranged in parallel rows, smaller ribs in parallel rows at right angles to and spanning between the girders, and a slab of uniform thickness supported by the two systems of ribs and girders, the whole being a monolithic construction. For long spans the floor slab is sometimes a flat-topped arch. For heavy loads and long spans the girders are also sometimes arched.

There are a large number of systems of reinforcement, some of which have already been described, and many of these may be applied to floor construction in one form or another. The multiplicity of systems make a description of them all not only impos-

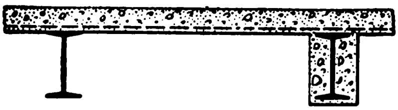


Fig. 293.—Monier Floor Slab Carried on Top Flange of Beam.

sible, but undesirable. A few of the principal systems will be used to illustrate their application to various kinds of building construction, and the reader should be able to apply the general principles here set forth to any of the systems not illustrated.

Filling Slabs Between Steel Beams.—The simplest form of reinforced concrete floor consists of a Monier slab resting upon the top flanges of the supporting beams, as shown in Fig. 293. Any one of the various types of reinforcement used for slabs and described in Chapter XIV. may be substituted for the Monier

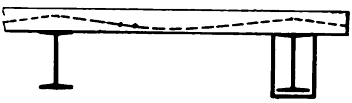


Fig. 294.—Floor Slab with Curved Expanded Metal Reinforcement.

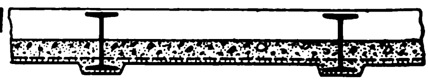


Fig. 295.—Monier Slab Carried on Bottom Flanges of Beams.

netting. When it is desired to protect the beam against fire, it is embedded in a mass of concrete, or three reinforced slabs may be used to box in the exposed faces of the beam, as shown in the right-hand portion of Fig. 294. The slab may rest upon the bottom flange, as in Fig. 295, and the space to the floor level filled with a weak cinder or coke concrete. The bottom flange of the beam is protected from fire by a layer of concrete. Sometimes both top and bottom slabs are used, with or without a meager concrete filling between them.

Expanded metal, the Clinton wire cloth and other slab reinforcements are used in the same manner as the Monier mesh, and may rest either upon the top or bottom flange of the beams. Fig. 295 is an example of expanded metal construction of this kind. The other systems are similar in all particulars. When a top slab is used an air space below the floor and protection for the bottom flange against fire is sometimes obtained by hanging a thin slab of concrete or plaster reinforced by expanded metal lath or some form of wire fabric from the bottom flanges of the beams. Both the Donath and Müller systems are used in the same manner. Figs. 105 and 106 show these systems resting upon the bottom flanges of the beams. Fig. 296 is a good example of an expanded metal slab with the reinforcement below the level of

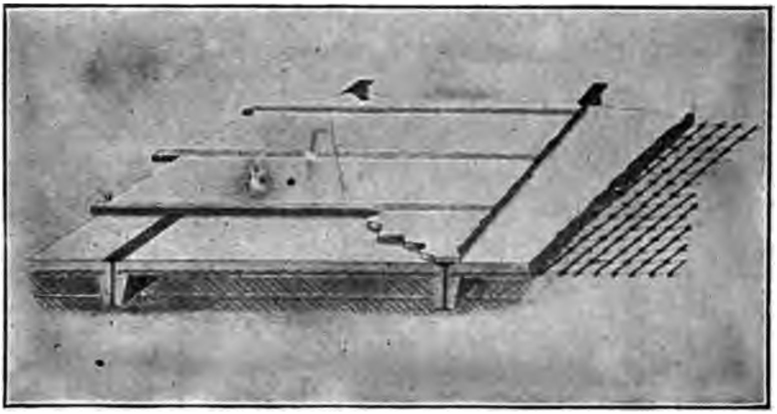


Fig. 296.—Floor Slab of Expanded Metal Carried on Lower Beam Flanges.

the top flange and supported by haunching resting upon the bottom flange of the beam. Expanded metal lath for a protecting ceiling is shown below the beams.

Monier slabs between beams are generally used in spans varying from 6 to 8 ft., and sometimes as great as 10 ft. The slabs vary in thickness, according to the loading, from 1½ to 4 ins. The carrying bars used vary in diameter from 3-16 in. to ⅜ in., and are spaced from 2 to 4 ins. centers; sometimes bars of larger diameter are placed at greater intervals. The spacing bars are usually from ⅛ to ¼ in. in diameter, and are spaced from 1 to 3 ins. apart. Care is taken when placing the carrying bars to have about ⅜ in. of mortar below them.

Expanded metal in this form of slab is used for spans up to 8 ft., this being the maximum length in which the sheet can be obtained. The thickness of the slab varies from 2 to 6 ins., and the meshing varies from 1 to 3 ins., the most common mesh used being 3 ins., made of No. 10 gauge metal.

The Müller system (Fig. 106) uses slabs up to 10 ft., with a thickness of from 3 to 6 ins. The carrying bars are flats, placed on edge, from 1 in. × 1/8 in. to 1 3-16 ins. × 3-16 in. in size, and the connecting bars are of sheet iron, the same width as the main bars and about No. 22 gauge, or 1-32 in. thick.

Fig. 297 shows one type of Roebling floor, which is similar to the Müller system. The reinforcement consists of 2 × 3-16-in. flat bars, usually spaced 16 ins. centers, embedded in cinder concrete. Several modifications of this general type are used for

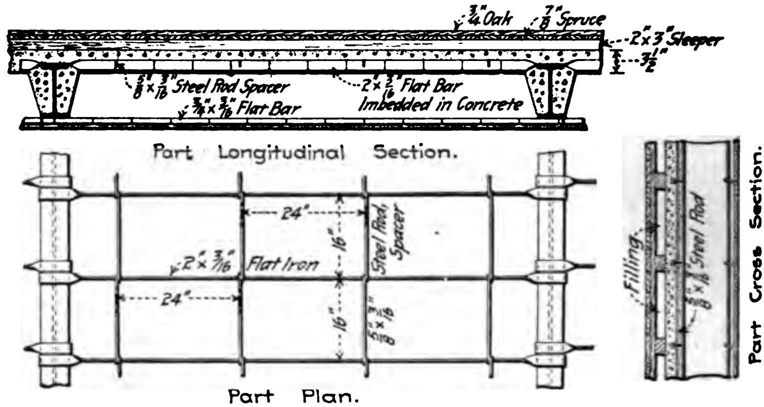


Fig. 297.—Roebling Flat Slab Floor.

spans up to 16 ft. and various loadings. The general style of construction will be understood from the figure.

The Donath slab (Fig. 105) is formed of small Tee or I-beams, varying from 3/4 to 1 3-16 ins. in depth and spaced up to 8 ins. apart. The transverse ties are of hoop iron. In this system a wire meshing is attached to the bottom of this framework and holds the concrete in place without any further centering. This slab is usually employed in spans of from 6 to 10 ft., and when the I-section is used up to 13 ft. The thickness varies from 3 to 6 ins.

The Columbia floor slab, described on page 237, consists of double-cross shaped bars suspended from the top flange of the beam by a stirrup, or, in case of the larger bars, connected to the web of the beam by riveted connection angles. The bars are spaced usually 24 ins. apart, but the spacing may change under varying conditions. The larger bars are used in spans up to 24 ft. This, however, is not an economic form of construction above about 16 ft. The thickness of the concrete slab varies with the

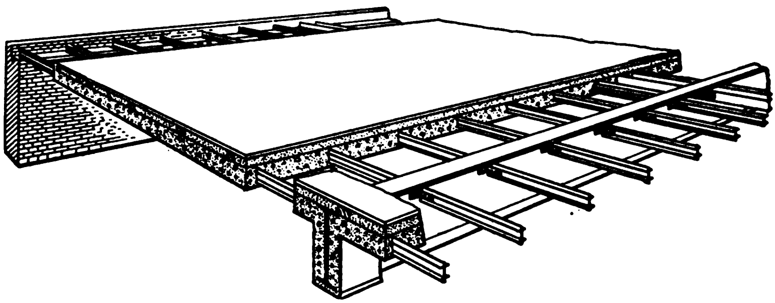


Fig. 298.—Columbian Slab Floor.

span, load and spacing. Fig. 298 shows details of this floor. Stone or slag concrete is employed for the supporting slab. A top dressing of cinder concrete, 2 ins. in thickness, is sometimes used, but no cinders are used in the main slab. The proportions of the mixture employed for stone concrete is usually 1 : 2 : 5,

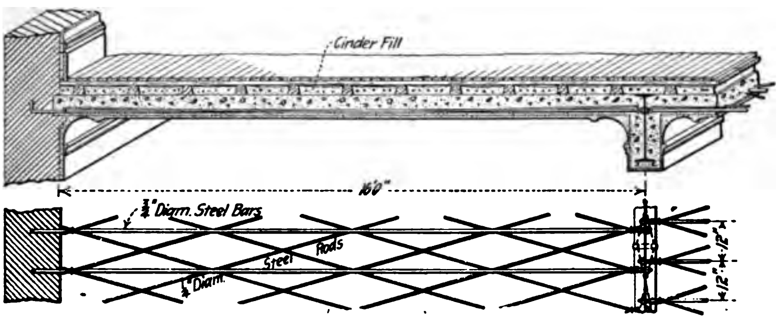


Fig. 299.—American Concrete Steel Co.'s Floor.

and for slag the proportions are usually 1 : 2½ : 3. A very wet mixture is used.

Fig. 299 shows the floor system used by the American Concrete Steel Co., Newark, N. J. This system is used in spans up to 16 ft. The reinforcement consists of ¾-in. diameter round bars, threaded at the end and strung between the webs of steel beams. They are held in place by nuts, and serve as tie rods as well as reinforcing rods. A secondary reinforcement of ¼-in. diameter rods is laced diagonally between the main rods and is wired to the latter at intersection points. The main rods are usually spaced 12 ins. centers. A cheaper form consists of using tee bars for the round rods, passing them over the top flange of the steel beams or resting them upon the bottom flange. The steel tees are sometimes hung from the top flange of the beam by means of stirrups similar to those used for the Columbian system. The ¼-in. secondary diagonal reinforcement is retained in all cases.

A modification of the floor construction described in the preceding pages consists in the use of a form of reinforcement similar to that explained in connection with Fig. 114. The reinforcement is located at the end of the slab, near the upper surface, and firmly anchored to the wall; it then passes to the lower part of the slab throughout its central portion; then again rises at the beam to the top of the slab, and so on throughout its length. Thus when the floor is continuous over several beams the condition practically amounts to fixing the slab at its points of support.

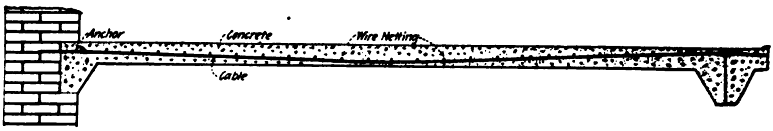


Fig. 300.—International Fence & Fireproofing Co.'s Floor.

Fig. 300 shows the floor section employed by the International Fence & Fireproofing Co., using tie-lock fabric, supported by wire cables for the reinforcement. The wire netting has a 6 x 6-in. mesh, and is supported by wire cables spaced from 12 to 18 ins. apart. This form of slab is used for spans of from 10 to 20 ft., and having thicknesses up to 6 ins. Care must be taken to firmly anchor the ends of the cables at the wall and girders.

The electric welded wire reinforcement is used in a similar manner, the carrying wires being large enough to take care of all tensile strain. This fabric has been used in spans up to 15 ft., with a thickness of slab of 6 ins.

Expanded metal may also be used in this curved form. When so used it is customary to lap the sheets at the middle of the slab. In this manner slabs of more than 8-ft. span may be obtained. Load tests on this type of expanded metal slab reinforcement have, however, not proved very satisfactory.

When adapted to the Monier netting the curved slab reinforcement is called the Koenen floor. This slab has been extensively applied by Mr. Koenen in Germany to the construction of floors for warehouses, factories, etc., in spans from 6.5 to 21.5 ft., with a slab thickness up to 8 ins. In the Koenen floor slab the rods are anchored by being bent up or fastened to transverse wall anchors at the ends. A mortar composed of 1 part cement to 4 parts sand is used by Mr. Koenen.

Fig. 301 shows another example of floor construction reinforced with ½-in. corrugated bars, and is adapted to spans up to 16 ft., with a thickness of from 3½ to 7½ ins.

Arched Floor Slabs.—Arched filling between beams has been extensively used in floor construction, especially when heavy loads

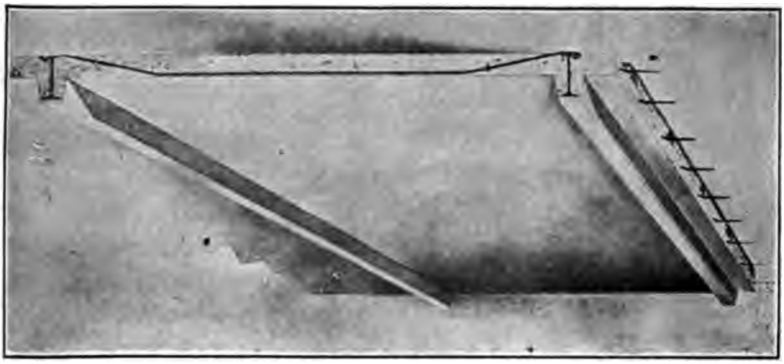


Fig. 301.—Slab Floor Reinforced with Corrugated Bars.

are to be carried. Fig. 302 shows a flat arch of expanded metal construction. These arches are used in spans up to 10 ft.

The Golding system is more frequently used than the form of reinforcement just described, and is well adapted to long spans. This system consists of a floor plate reinforced with expanded metal. The slab is strengthened with concrete arch ribs, spaced from 4 to 6 ft. apart, the rib resting upon and being reinforced by soffit channels laid flat, with their backs down; 6-in. channels, weighing 12¼ lbs. per ft., are generally used. The channels are sometimes exposed, but are usually surrounded with metal lath and plastered to protect them from fire. The Golding paneled floor is used in spans up to 20 ft. Fig 157 shows this form of arch flooring.

The Roebling system of floor arching consists of a wire cloth centering, stiffened by steel rods woven into the mesh; 7-16-in. diameter rods are used for 5 ft. and 9-16 in. for 7-ft. spans. This centering is sprung between and rests upon the bottom flanges of the floor beams. The concrete is deposited upon this wire centering with its top surface flush with the top flanges of the

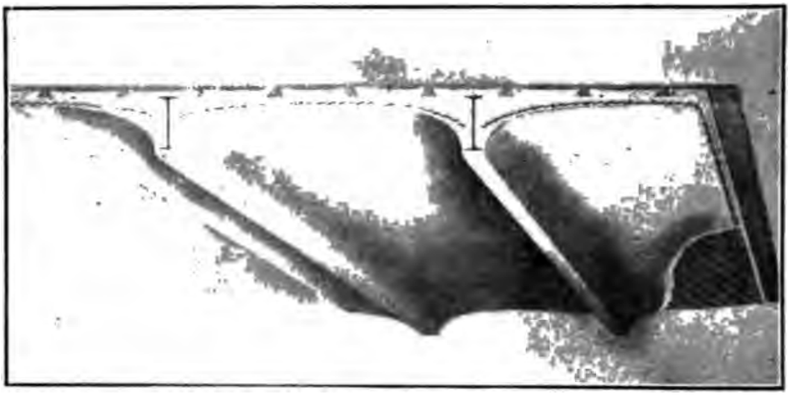


Fig. 302.—Flat Arch. Expanded Metal Construction.

beams. When a flat ceiling is not desired, the bottom flanges of the beams are enveloped with wire cloth and the whole embedded in concrete. When a flat ceiling is required, a clip of special form is attached to the lower flange of the beams and supports flat iron bars set on edge and spaced about 16 ins. centers. Roebling standard wire lath, with ¼-in. steel stiffening ribs woven in,

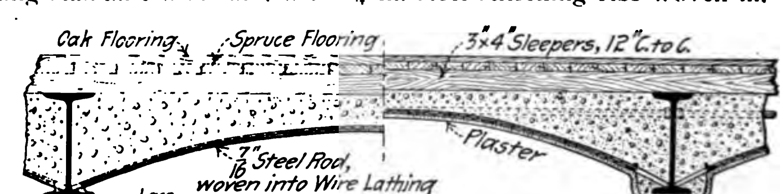


Fig. 303.—Roebling Arch Floor.

is laced to these ceiling bars. The whole is plastered over to form the ceiling.

This form of construction is commonly used for factories, warehouses, etc., in spans up to 7 and 8 ft., and will sustain loads of 4,000 lbs. per sq. ft., and can be adapted to loads of 10,000 lbs. per sq. ft. A modification of the above system, in which the stiffening rods are replaced by T's, is used in spans up to 16 ft.

The T-irons are usually spaced 2 ft. centers, and the wire cloth centering laid between the T-iron ribs. Fig. 303 shows the usual type of Roebling floor arch. The thickness of the arch at the crown varies from 2 to 4 ins. The concrete mixture used for Roebling floors is 1 cement, 2 sand and 5 cinders. A 1 : 2½ : 6 mixture is also sometimes used.

Fig. 304 shows the arched type of floor slab used in the construction of the first floor and sidewalk of the Metropolitan Building, New York City, while Fig. 305 shows the flat floor slab used for the upper stories. De Man bars, described on page

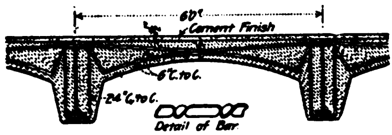


Fig. 304.—De Man Arch Floor.

227 form the reinforcement, and were made of 1 × ⅛-in. steel strap, crimped at regular intervals. The location of the reinforcement will be understood from the drawing. The bars are spaced from 6 to 12 ins. apart, according to the floor loading. The American Fireproofing Cement Construction Co., of New York City, build this type of floor.

When considerable strength is needed Monier arches are used in place of Monier slabs. They usually consist of an arch having a rise of $\frac{1}{10}$ the span, springing from the lower flanges of the

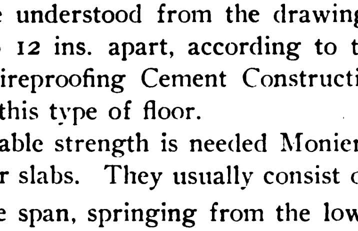


Fig. 305.—De Man Flat Floor.

supporting beams. A single Monier netting, placed near the introdosal face, constitutes the reinforcement. A filling of meager concrete fills the intervening space up to the floor surface. The bottom flange of the beam is protected by a layer of concrete about 1½ in. thick. Mr. Wayss, an Austrian engineer, states that it is customary to use Monier arches up to 5 m. (16.4 ft.) span, with a single netting and a thickness of arch ring of 5 cm. (2 ins.) to carry loads up to 1,200 kg. per sq. m. (230 lbs. per sq. ft.). Fig. 306 shows examples of single and double Monier floor slabs. Sometimes a second reinforcing mesh is used. It may be employed in a second arch ring some distance above the first, with a filling of meager concrete between, but the most common form is to place it near the surface of a flat extrados. Arched floors, with a flat extrados up to 6 m. (20 ft.) span, were used in the construction of Public Buildings at Kameroun. Fig. 307 shows an example of an arch with a reinforced flat extrados which was used in the construction of a warehouse at Trieste.

The Melan system (Fig. 308) employs rolled beams for the

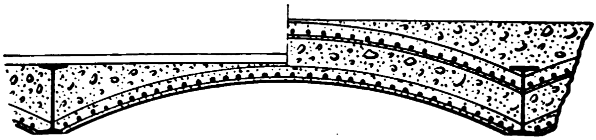


Fig. 306.—Single and Double Arch Monier Floor.

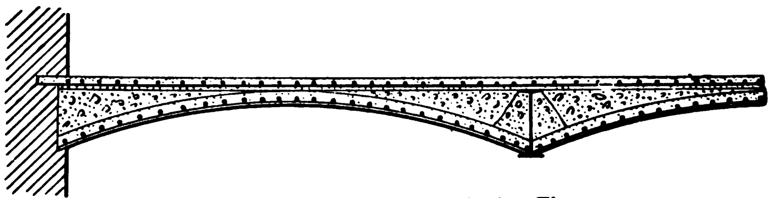


Fig. 307.—Flat Top Arch Monier Floor.

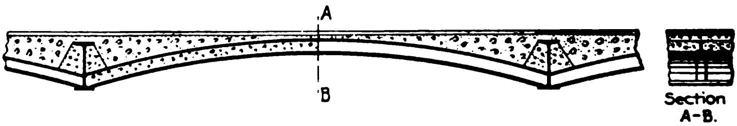


Fig. 308.—Melan Arch Floor.

Fig. 309.—Wunch Arch Floor.

reinforcement, and is most frequently used in spans of from 8 to 14 ft., with a thickness of arch ring of 3 ins. The rise is usually from $\frac{1}{14}$ to $\frac{1}{10}$ of the span. The ends of the beams are mitered to bear against the web of the supporting beams, and rest directly upon the bottom flanges. This is not an economic form of construction unless unusually heavy loads are to be carried.

The Wünch system (Fig. 309) uses angles or T's for reinforcement. A double reinforcement is used with the ends of the angles or tee irons riveted to the flanges of the beams. This gives a rigid skeleton work, but is expensive.

Monolithic Floors.—The various systems of floor slabs just described, with few exceptions, may be increased in size to form monolithic floors of considerable span. While these various constructions may be increased to quite long spans, when so used they are not economical, and hence are not used except in cases where expense is of secondary importance, as when the demand for clear floor space prevents the use of columns, or when limited head-room makes the extra depth necessary for ribs undesirable. When the floor panels can be built in the form of squares or rectangles, with their sides of nearly equal length, and are supported

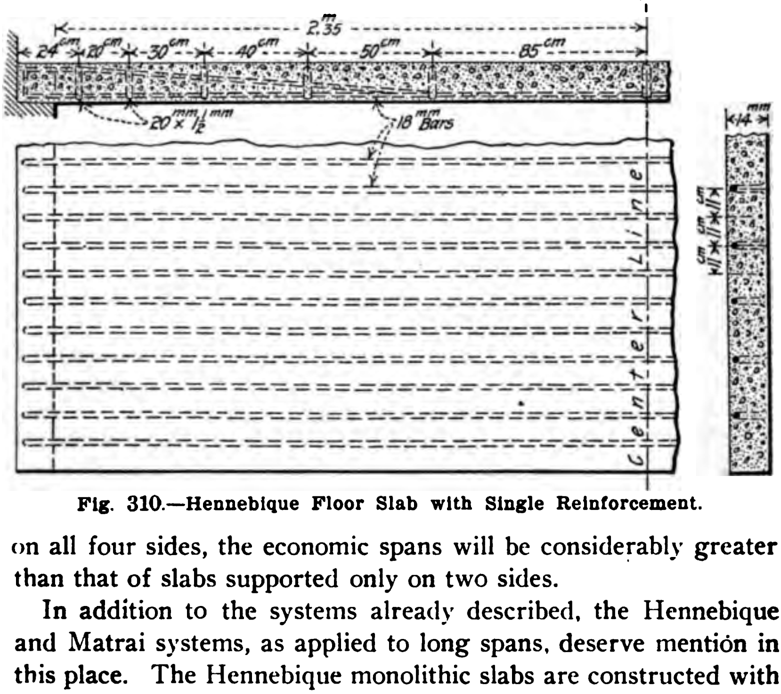


Fig. 310.—Hennebique Floor Slab with Single Reinforcement.

on all four sides, the economic spans will be considerably greater than that of slabs supported only on two sides.

In addition to the systems already described, the Hennebique and Matrai systems, as applied to long spans, deserve mention in this place. The Hennebique monolithic slabs are constructed with three forms of reinforcement; first with single independent bar reinforcements, consisting of alternate straight and bent round rods spaced at equal intervals. Fig. 310 shows the principal characteristics of this form. The rods have their ends split to anchor them in the concrete. The size and spacing of the rods depends upon the span and load. They usually have a diameter of from $\frac{5}{16}$ to $\frac{3}{4}$ in. and are spaced from 4 to 12 ins. centers. When the thickness of the slab is greater than 3 ins. stirrups are employed. The stirrups are of hoop iron, about $\frac{3}{4} \times \frac{1}{16}$ in., and extend up into the slab to within about $\frac{3}{8}$ in. of its top face. The example shown in Fig. 310 was used in the construction of a banquet hall at Basel, Switzerland.

The second form of Hennebique slab has a series of straight rods at right angles to the first series, with alternate rods placed near the top and bottom of the slab. This form and the one yet to be described are more suitable for reinforcing square floor slabs and rectangular slabs having their dimensions practically equal. Stirrups are only placed about the lower bars. In proportioning these slabs they are considered as resting upon four supports.

Fig. 311 shows this type of slab. The third form of Hennebique floor slab has a lattice reinforcement made up entirely of

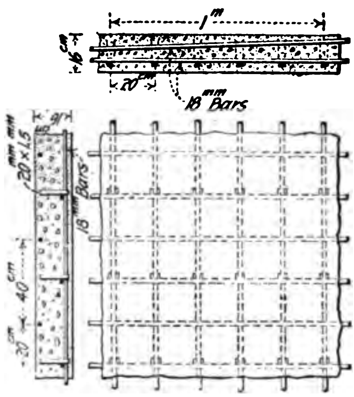


Fig. 311.—Hennebique Floor Slab with Double Reinforcement.

straight rods, with alternate rods placed near the top and bottom of the floor plate. The bottom bars only are provided with stirrups.

In the construction of the Matrai system of floors the principle explained in regard to slab reinforcement on page 244 and illustrated in Figs. 115 and 158, is extended, and develops very complicated systems of wire netting. When used in connection with beams and girders the wires are attached as near as possible to the ends of the beams and girders, in order that their bending moments may be reduced as much as possible and their sections correspondingly reduced. The space to be spanned is divided into rectangular panels by reinforced concrete girders, whose reinforcement consists of stiffening skeleton work, such as is shown in Figs. 91 and 92. These are strengthened by wire cables hung in the form of a catenary. Sometimes cables are strung diagonally through the panels, as shown in Fig. 312. This figure

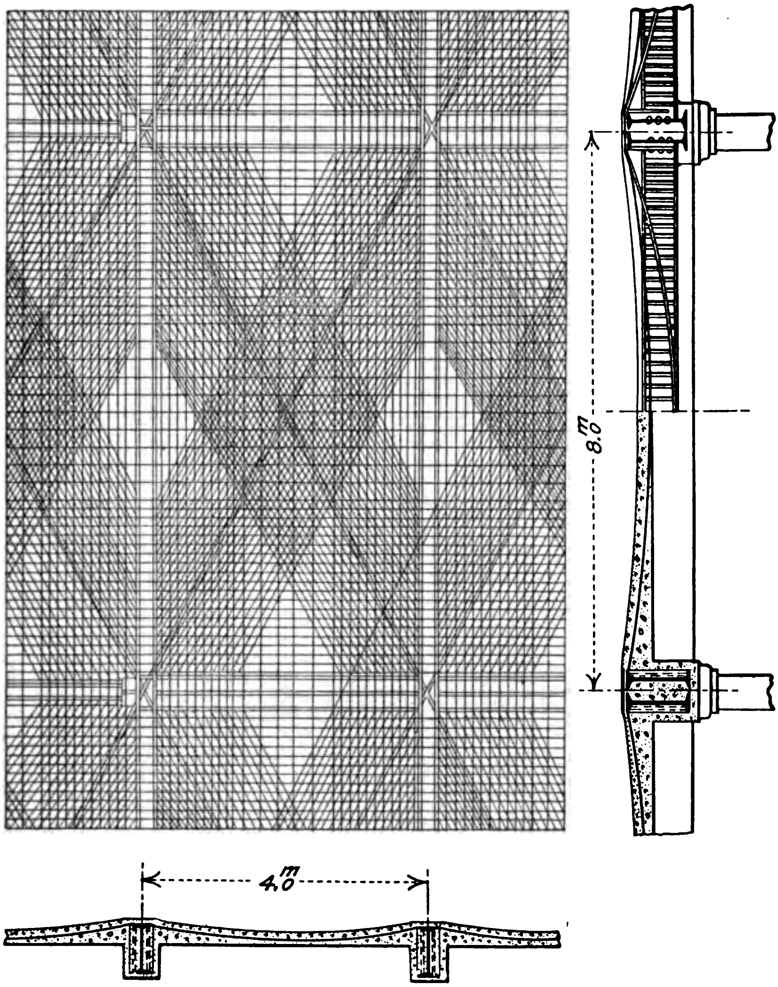


Fig. 312.—Matrai Floor.

shows the section of a floor used in the construction of the Maison d'Education de la Legion d'Honneur. The concrete used in this floor was 1 part cement, 2 parts sand and 4 parts slag. The concrete acts simply as a filling and protecting material in the Matrai floor system, reliance being placed alone upon the wires and cables to carry the loads.

The Mushroom System of Construction.—This system is the invention of Mr. C. A. P. Turner, M. Am. Soc. C. E. By referring to Figs. 313 and 314, it will be seen that the construction consists of columns, and a floor slab without beams or girders. The mushroom action, so called, is obtained by arranging the reinforcing metal as shown in Fig. 314, the top of the column being enlarged to form a capital, as shown in elevation, Fig. 313. The reinforcing rods for the slabs are then strung, as shown in Fig. 314, in a manner similar to that used in the Matrai system,

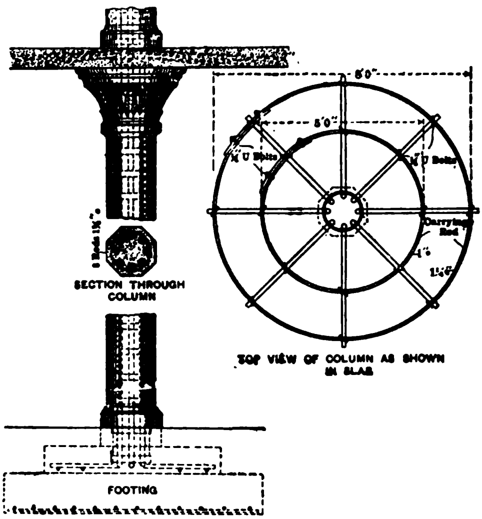


Fig. 313.—Column in Mushroom System of Construction.

as shown in Figs. 115 and 312, which this system in many ways resembles. The arrangement of the reinforcement, as used in the construction of the C. A. Bovey Building, Minneapolis, Minn., is shown in Fig. 315.

The advantages claimed for this method of construction are a reduction in the cost of forms, owing to the flat ceiling used, and an increased strength due to mushroom shape at top of columns. It would appear that an analysis of the stresses in the floor slab would be a rather uncertain and puzzling operation. Mr. Turner states that this system has been successful in competition with wood mill building construction.

Ribbed Slabs.—The most common form of floor construction

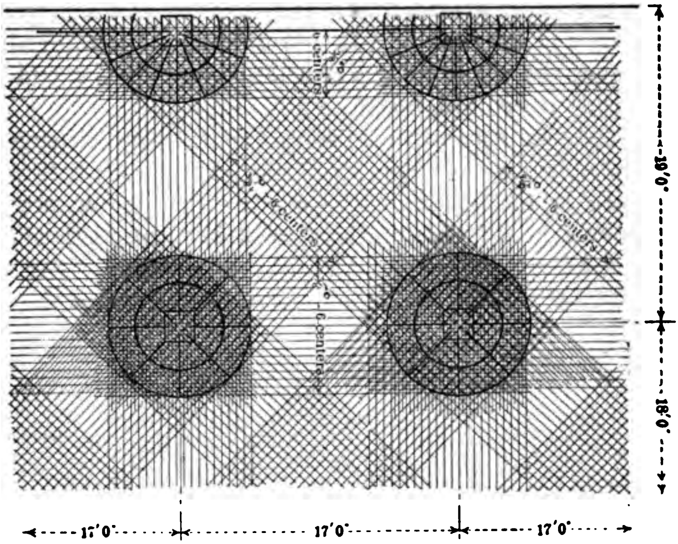


Fig. 314.—Floor Slab Reinforcement in Mushroom System of Construction.

Fig. 315.—View of Floor, Mushroom System of Construction.

when the spans are greater than 6 or 8 ft. is that of a concrete slab strengthened at intervals of 4 ft. and upwards by reinforced concrete beams, the whole being a monolithic construction and commonly called ribbed slab floors.

In ribbed slab construction the various systems are employed in a similar and often identical manner, usually the only difference being in the manner of reinforcing the slab or beam, or both. These have been fully explained in connection with descriptions of reinforcements for slabs and beams. For short spans and light loads the plain floor slab will prove the most

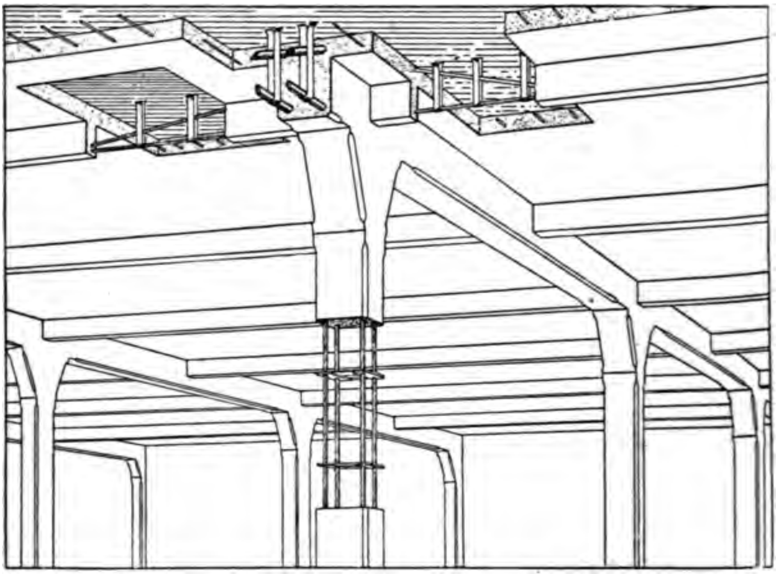


Fig. 316.—Typical Column and Floor Construction by Hennebique System.

economic, especially as the necessary forms are more simple than those used in the construction of ribbed beams. When the spans become somewhat greater, however, the ribbed slab will prove the more economic. The spacing of the ribs may be varied in order to secure a paneled effect for the ceiling. In fact, in this respect reinforced concrete construction admits of great flexibility in this regard, and often enables the architect to greatly improve the beauty of the structure.

When a flat ceiling is desired, a suspended ceiling may be used similar to that described on page 483. The Hennebique system has been extensively applied to the construction of floors. The floor slabs used are from 2½ to 6 ins. thick, and are, when possible, made continuous over the ribs or reinforced concrete girders. Fig. 133 shows a typical form of Hennebique floor construction. Two systems of ribs are employed. The main ribs are usually

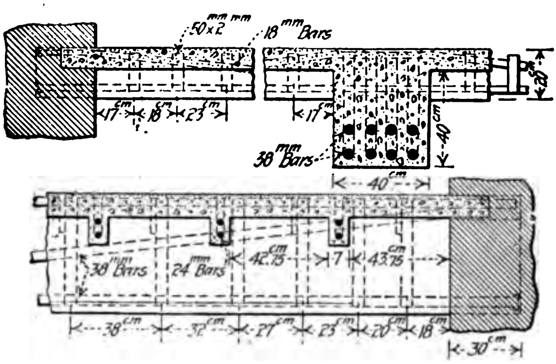


Fig. 317.—Floor for Palais de Justice, Viviers, France.

limited to spans of from 16 to 24 ft., although beams with a span of 60 ft. have been built. The secondary ribs are generally not greater than from 10 to 12-ft. span, and are usually reinforced with a single rod or pair of rods and with stirrups, which are placed about the bottom rods only. The floor space is divided

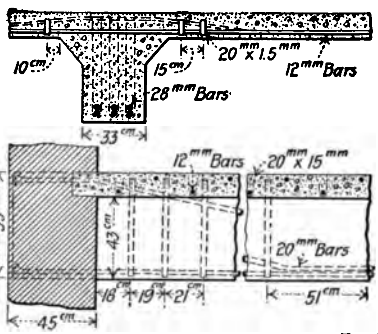


Fig. 318.—Floor Petit Palais des Beaux Arts, Paris, France.

into square or rectangular panels by these two systems of beams. The reinforcing rods used vary from ¼ in. to 2 ins. in diameter, and are spaced so that there will be at least from 1¼ to 2½ ins. of concrete between them, and the rods should not be nearer than within 1 in. of the lower face of the beam. Fig. 316 shows the general features of a Hennebique floor and column construction.

M. Christophe gives excellent examples of this form of construction in his book, "Beton Armé" (see pages 109-111), from which the following figures are taken. The first of these (Fig. 317) is a section of floors used in the construction of the Palais de Justice de Viviers, France. A main rib, heavily reinforced, secondary ribs and floor slab are all used together in this building. Sometimes the secondary ribs are omitted, and we have a con-

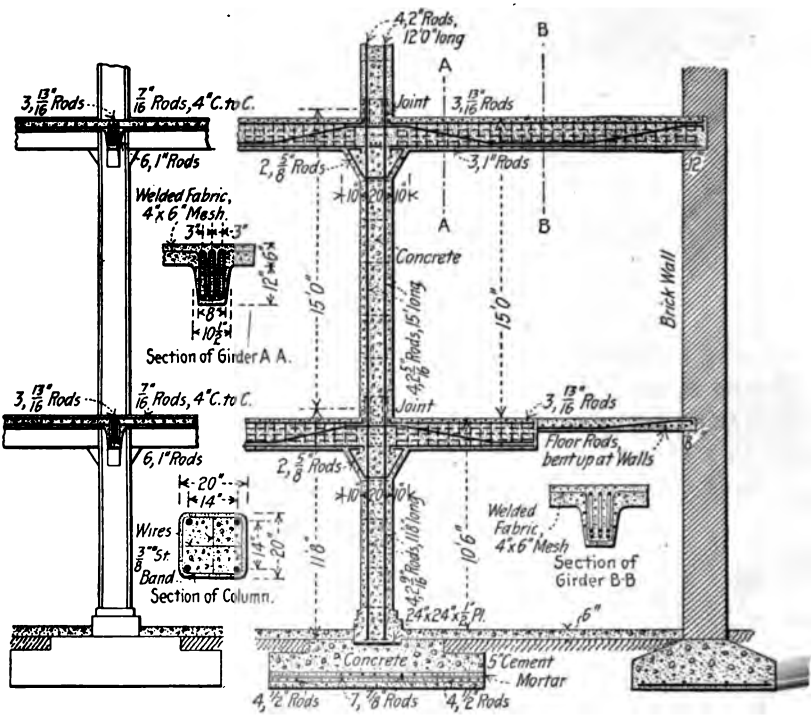


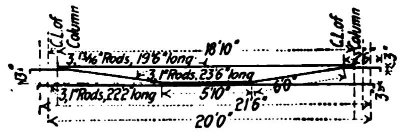
Fig. 319.—Floor for Chicago Store Building.

struction like that shown in Fig. 318, which is a section of the floor used in the Petit Palais des Beaux Arts at the Paris Exposition of 1900. The floor has a span of 7.35 m. (24.11 ft) for the ribs, which are spaced about 7.5 ft. centers.

A modification of the Hennebique construction is shown in Fig. 319, which shows the details of a reinforced concrete store building in Chicago. The reinforcing rods used in the floor girders were ¾ and 1 in. in diameter, both tension and compression rods being employed.

Fig. 320 shows the rods used to form the truss of each girder. These rods are connected by an electrically welded fabric netting, which envelops each girder. The ends of the rods are bent up at right angles to firmly anchor them in the concrete. The main floor rods are $\frac{7}{16}$ in. in diameter and spaced 4 ins. centers in both directions and are carried through the girders. A 1 : 2 : 2 concrete of Vulcanite Portland cement, sharp torpedo sand and gravel screened to pass a ⅝-in. mesh was used.

Mr. E. T. Ransome has erected a large number of buildings in



Reinforcing Rods in Girder.

Fig. 320.—Girder Reinforcement, Chicago Store Building.

this country. Twisted rods are exclusively used by this engineer for reinforcing beams, columns, slabs, etc. In the construction of ribbed floors the ribs are usually reinforced with one or more straight rods in the tension flange. Stirrups of square-twisted rods tie the reinforcement to the concrete and are spaced close together at the ends and further apart toward the center as the shear decreases.

Fig. 321 shows a section of a floor in an addition recently made to the reinforced concrete building erected in 1897-8 for the

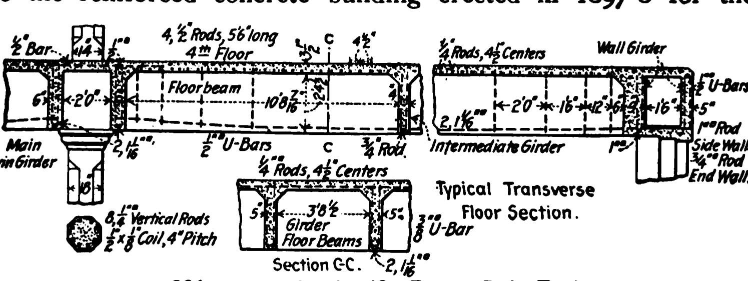


Fig. 321.—Floor for Pacific Borax Co.'s Factory.

Pacific Coast Borax Factory, Bayonne, N. J. The floors in this addition, of which the above figure is a representative example, are designed for a dead load of 100 lbs. and a uniformly distributed live load of 400 lbs. per sq. ft. The transverse beams are 4 ft. 1½ ins. on centers and span between columns spaced 24 ft. 8⅞ ins. on centers. Twin girders are used at the columns, and are separated by a cleavage plane to allow for expansion. The floor slab is 3½ ins. thick and reinforced by ¼-in. twisted steel longitudinal bars spaced 4½ ins. centers.

Another notable example of Ransome construction is that of the Kelley & Jones Co.'s concrete-steel factory building, Greensburg, Pa. This building is 60 × 300 ft. in plan and four stories high. Fig. 322 shows traverse and longitudinal sections of the floors, walls and roof of this building. The first floor is of concrete and rests directly upon rammed earth. The upper floors

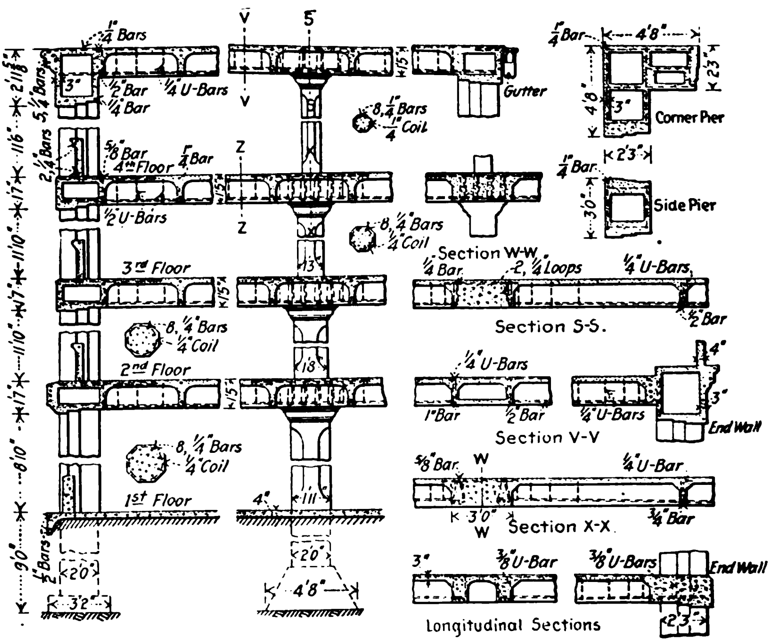


Fig. 322.—Floor and Wall Details, Kelly & Jones Factory.

are designed to carry 250 lbs. per sq. ft., and are divided into panels about 3 ft. 8 ins. wide and 8 ft. 4 ins. long by transverse and longitudinal girders. The 3-in. unreinforced floor slab is supported by 12-in. longitudinal and transverse girders. The main transverse girders are in pairs, supported on each row of columns, and carry 13 lines of longitudinal beams connected by transverse girders at the center of each panel. The longitudinal beams are spaced about 3 ft. 10 ins. centers, are 3 ins. wide, and reinforced with one 1⅛-in. bar in the lower side and by one ¼-in. bar in the upper part of the floor slab. Vertical stirrups of ½-in.

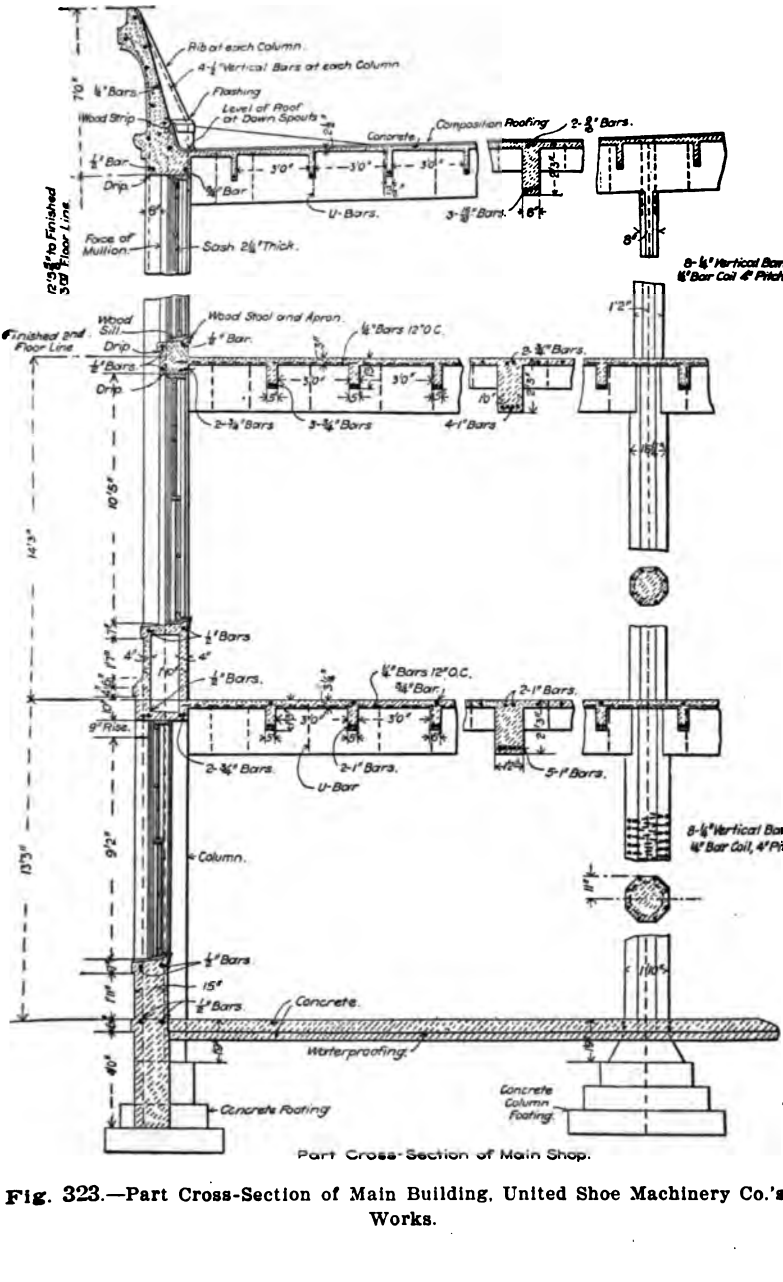


Fig. 323.—Part Cross-Section of Main Building, United Shoe Machinery Co.'s Works.

steel bars are used, and are spaced close together at the ends of the beams and further apart near the middle, being spaced to conform with the strain sheet of maximum shears and relieve the concrete of all the computed shearing stress. The main transverse girders are 9 ins. wide and reinforced with three $1\frac{1}{8}$-in. rods, and are arranged in pairs 9 ins. in the clear from the center line of the columns. A feature of this building is the absence of reinforcement in the floor slab.

The above two examples serve to illustrate the earlier types of reinforced concrete factory building constructed in this country.

The shops of the United Shoe Machinery Co., Beverly, Mass., consisting of ten buildings, covering about four acres, are probably the largest single reinforced concrete building construction ever undertaken in this country. The floor space in these buildings is about 18 acres. The total estimated cost of these buildings exclusive of land, which, with the exception of the roofs of the foundry and forge shops, are entirely of reinforced concrete, is about $1,000,000.

The Ransome system of twisted steel bars is used for reinforcement. The structural concrete was all mixed 1 : 2 : 4, with screened gravel from $\frac{1}{4}$ to 1 in. in diameter; Lehigh and Atlas brands of cement were used and the concrete was mixed wet. The buildings were divided into sections 60 ft. in length, and all concrete in each section was built as a monolith. The two largest buildings are 62 ft. wide, 522 ft. long and four stories in height. Fig. 323 is a partial cross-section of one of the main buildings, and shows the type of construction used throughout all the buildings. Fig. 324 shows partial plan of first floor at one end. The arrangement of girders and beams will be understood from the plan. It should be noted that the corner panels are monolithic slabs without floor stringers, and have the reinforcing bars crossing the panel in a diagonal direction. This construction stiffens the building throughout the planes of the floors against distortion due to wind pressure, and is used for all floors and the roof at the ends of the buildings. The size and spacing of rods in slabs, beams and girders, as well as general details of construction, are shown in the figures. The roof was designed for a live load of 75 lbs. per sq. ft., the third and fourth floors for 200 lbs., and the second floor for 250 lbs. per sq. ft.

Fig. 325 is an excellent example of ribbed floor construction as used in factory building. The live load it was designed to carry is 200 lbs. per sq. ft. The longitudinal beams are spaced 3 ft. 9 ins. centers, and are carried by transverse girders 12 to 15 ft. apart on centers and from 15 to 16 ft. in length. The concrete used in the construction of this floor was a 1 : 2 : 4 Edison Portland cement, sand and trap-rock crushed to pass a ¾-in. screen. This floor was used in the construction of the Thompson & Norris eight-story factory building, in Brooklyn, N. Y. Messrs. H. C. Miller and H. I. Moyer were the engineers in charge of the design and construction of this building.

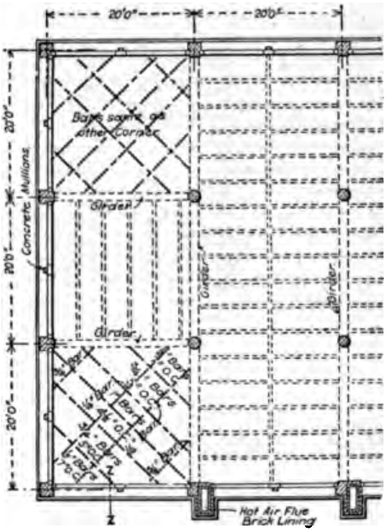


Fig. 324.—Part Floor Plan, United Shoe Machinery Co.'s Factory.

A form of ribbed floor construction used by the Reinforced Cement Construction Co., New York, is a modification of the de Vailliere system, and consists of straight and round rods similar to those used in the Hennebique system for the beam reinforcements and straight rods at right angles to these for the slab reinforcement. The lower straight rods are rigidly connected to the slab reinforcement with twisted rod stirrups, causing the two to act together as a tee-beam. Fig 326 shows the arrangement of rods and stirrups. The stirrups are sometimes inclined, upon the supposition that they will thus better care for the shearing stresses. Fig. 327 shows the method in which this system is applied to column, girder, beam and slab construction.

This system was used in the construction of the Hugh Bilgrim Machine Shop, Philadelphia, Pa. This building is 120 × 100 ft. in plan, and consists of a main building, 120 × 53 ft., five stories high, and a one-story extension occupying the remaining space.

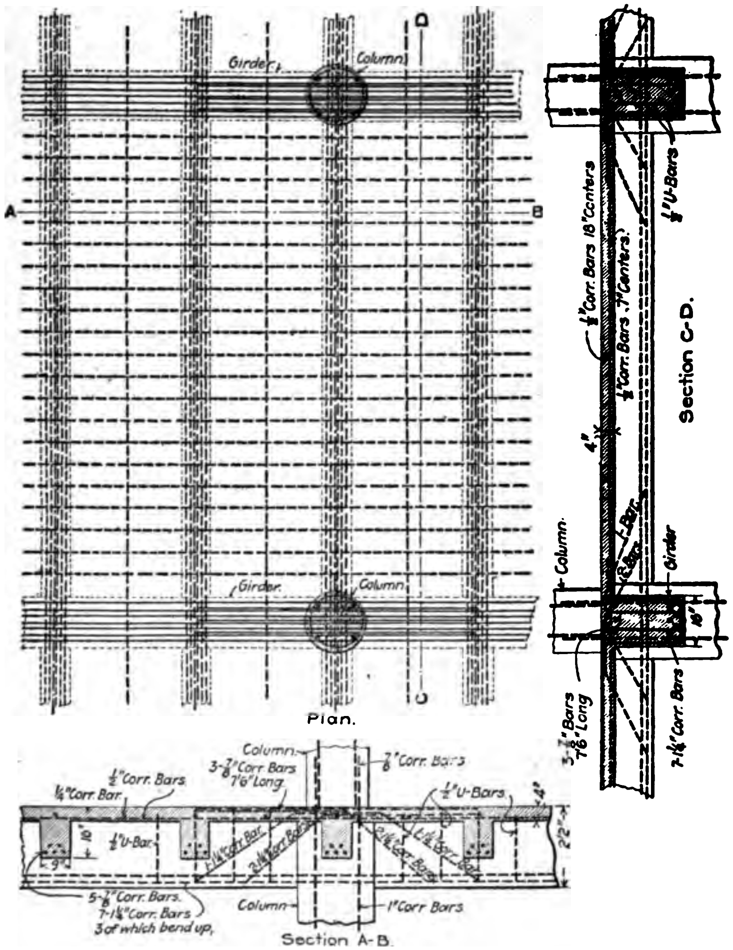


Fig. 325.—Floor Construction for Thompson & Norris Factory.

The columns were spaced 18 ft. 6 ins. centers transversely and 14 ft. 4 ins. longitudinally. The transverse beams supporting the floor are 6 × 12 ins. in section and spaced 3 ft. 7 ins. centers. The floor slab is 4 ins. thick on the first floor and 3 ins. on all other floors. Girders 10 × 14 ins. in section are supported by the columns and carry the transverse floor stringers. Fig. 328 shows the arrangement of the reinforcement for the floor slab, girders and columns. The column sections are 21, 19, 17, 13 and 8 ins. on the first, second, third, fourth and fifth floors, respectively. The first floor was designed for a uniform live load of 300 lbs. per sq. ft., the second floor for 200 lbs. and the other floors for

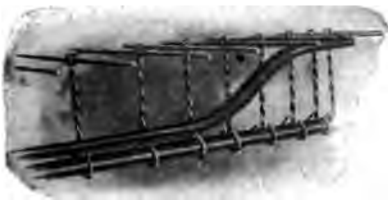


Fig. 326.—Girder and Slab Reinforcement, Reinforced Cement Construction Co.'s System.

150 lbs. per sq. ft. Round rods with twisted rod stirrups, as shown in Fig. 326, were used for reinforcements. The concrete mixture was 1 part Alpha Portland cement, 3 parts clean sand and gravel and 5 parts trap-rock, broken to pass a ¾-in. ring. Mixing was done by hand. The saw tooth roof used on the rear part of this building is described on page 548.

A good example of the adaptability of reinforced concrete to

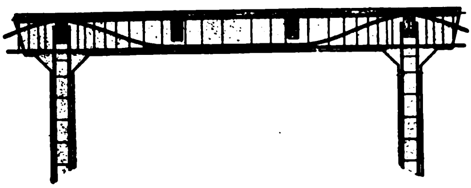


Fig. 327.—Column and Floor Construction, Reinforced Cement Construction Co.'s System.

the construction of a building where wide floor space is desired is that of the Robbins automobile garage, in New York City. This is a three-story and basement structure, with reinforced concrete floors and columns and brick enclosure walls. The building is 50 × 100 ft. in plan, with an ell 32.5 × 36 ft. in plan on one side at the rear. The floor space on all three stories is entirely unobstructed. The concrete floor slab of the first story rests on beams and girders carried by reinforced concrete columns built up from the basement floor, while the slabs of the second and third floors and the roof rests on heavy transverse girders which are carried at each end by the reinforced concrete columns built in the outside brick walls. This arrangement is shown by Fig. 329, which is an interior view of one floor. Fig. 330 is a section of one of the main girders, showing details of the reinforcement. In the main part of the building the basement is divided into three bays by the outer walls and two rows of 12 x 12 columns. The

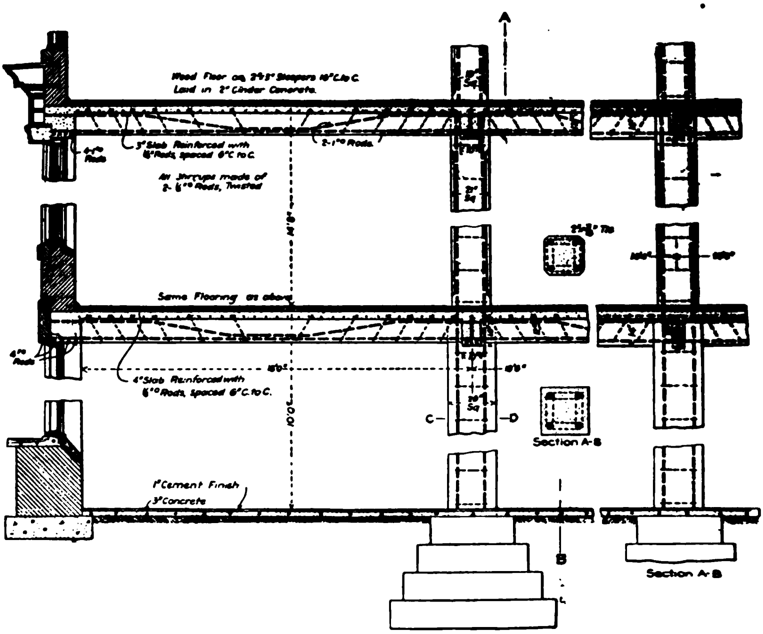


Fig. 328.—Part Section of Hugh Bilgram Machine Shop, Reinforced Cement Construction Co.'s System.

columns are spaced 13 ft. 5 ins. centers longitudinally and 17 ft. 3 ins. centers transversely. They carry 6 x 14-in. transverse beams and 8 x 18-in. longitudinal girders, which are built monolithic with the floor slab. The second and third floors are carried by 20 x 36-in. transverse girders, spaced 13 ft. 5 ins. centers. These girders are 50 ft. long, with a clear span of 45 ft., and are built monolithic at the ends, with 24 x 26-in. columns carried up from the basement floor. The floor slab between the girders is carried by 6 x 14-in. cross beams 6 ft. 3 ins. centers. The roof girders and beams are lighter than the floor slabs, as they are designed for lighter loads. The slabs of the floor and roof are 5 ins. thick. Round rods were used for the reinforcement throughout and arranged according to the de Vailliere system.

Fig. 329.—Interior View, Robbins Garage, New York City, Showing Floor Girders and Columns.

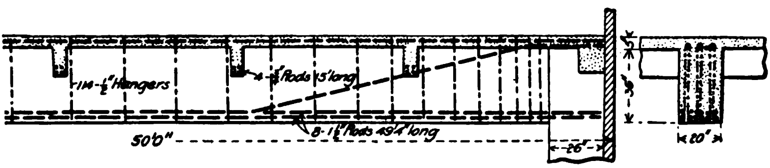


Fig. 330.—Section of Main Girder in Robbins Garage, New York City.

A wet concrete was used in proportions of 1 part Vulcanite cement, 2 parts sand and 3 parts of ¾-in. broken stone with the dust screened out.

The forms were made of 2-in. plank, dressed on one side. Those for the floor and roof slabs were made of tongue and grooved flooring.

Another reinforced concrete building in which the de Valliere system of reinforcement is used by the Reinforced Cement Construction Co. is that of the paint and overhauling shop of the Philadelphia Rapid Transit Co. This building is two stories in height and 90 × 389 ft. in size, the first floor having a clear height under beams of 18 ft., and the second floor a clear height at eaves of 15 ft. 2 ins. Five lines of tracks are provided for on the first floor and six on the second floor. To facilitate handling of cars a transfer table was placed near the rear of the building on both floors in place of the ladder tracks usually employed for entering the various floor tracks from the street. This necessitated girder spans of 48 ft. for supporting the floors. The transfer table is carried on four lines of rails, supported on 18 x 18-in. cross beams carried by longitudinal girders 18 x 39½ ins. in section, having a clear span of 46 ft. between columns. These girders are joined to the columns by special brackets. The columns and girders are built together and form a monolithic structure. The general

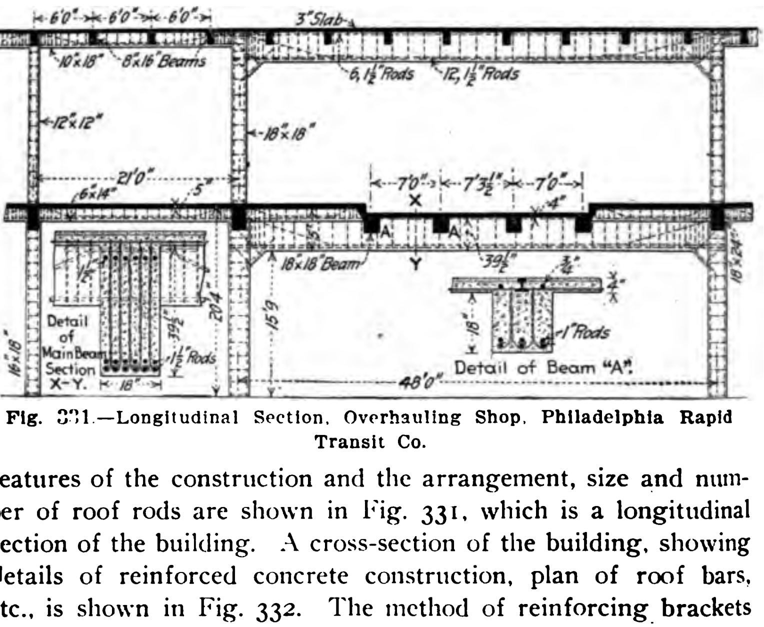


Fig. 331.—Longitudinal Section, Overhauling Shop, Philadelphia Rapid Transit Co.

features of the construction and the arrangement, size and number of roof rods are shown in Fig. 331, which is a longitudinal section of the building. A cross-section of the building, showing details of reinforced concrete construction, plan of roof bars, etc., is shown in Fig. 332. The method of reinforcing brackets on columns in the first story to support the crane girder is also shown in Fig. 332.

Another form of ribbed slab construction, used for both roof and floor slabs, was employed in the construction of the Central Felt & Paper Factory, at Long Island City, and of the Parkville sub-station of the Brooklyn Rapid Transit Co. The reinforcement consists of a straight rod and one or more bent rods in the same vertical plane, there being two or more series of rods parallel

:o each other, and all wired together to make up the skeleton work ɔf one. girder.

This makes quite a stiff framework, which is hung in the

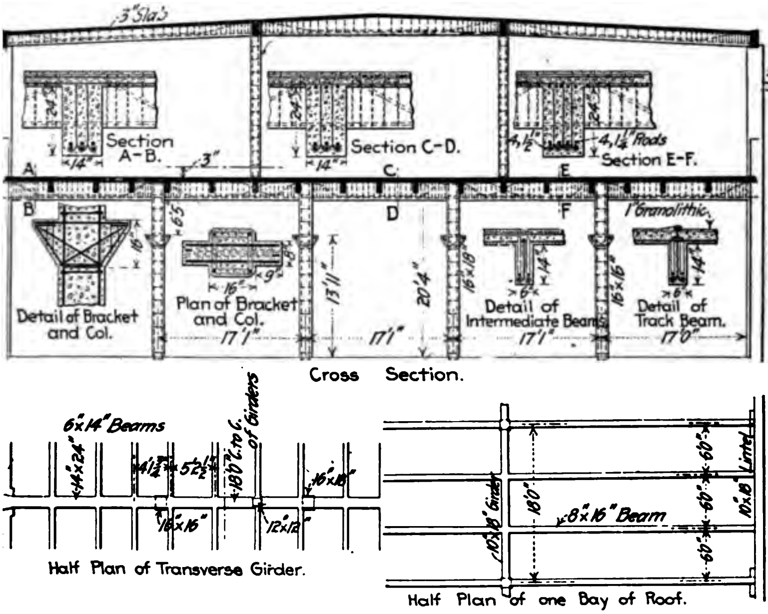


Fig. 332.—Transverse Section, Overhauling Shop, Philadelphia Rapid Transit Co.

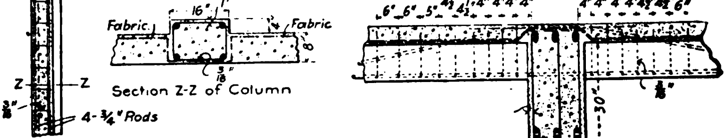


Fig. 333.—Roof Girder, Central Felt and Paper Co.'s Factory.

orms, and the concrete is tamped firmly about the metal. It is laimed that the metal when thus used will not be displaced when ıe concrete is put in. Compression rods are also sometimes used.

The floor slab is reinforced in the usual manner with a series of parallel rods. The connecting rods or stirrups are spaced to correspond to the diagram of shears. Fig. 333 shows the details of a 52-ft. roof girder used in the construction of the Central Pulp & Paper Co.'s building. Wire fabric was used for reinforcing

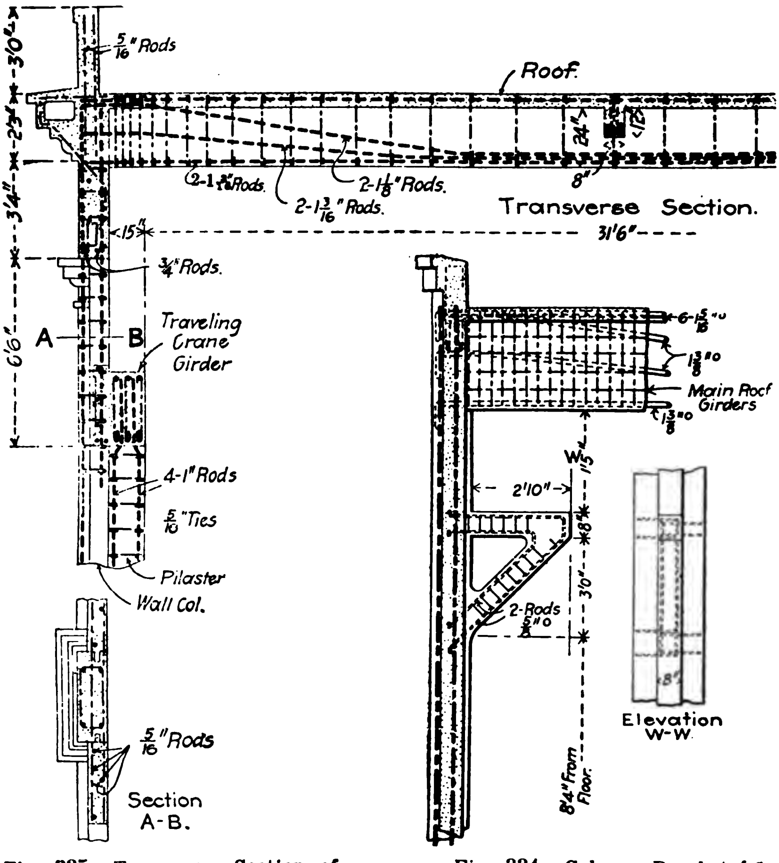
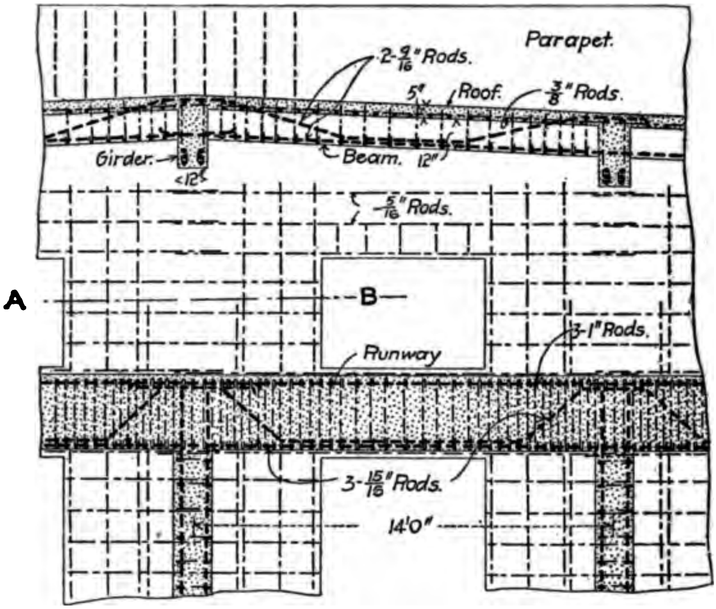


Fig. 335.—Transverse Section of Sub-station Building, Brooklyn Rapid Transit Co.

Fig. 334.—Column Bracket for Crane Run Girders, Central Felt and Paper Co.'s Factory.

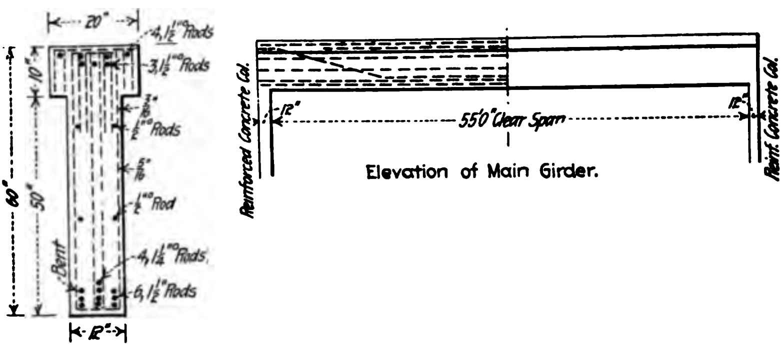
the roof slabs. Fig. 334 shows details of wall bracket used for carrying crane-run girders in this building. Fig. 335 shows a transverse section of wall and roof construction, and Fig. 336 longitudinal sectional elevation of Parkville sub-station, together with framing of roof and crane-run girders.

A long span girder used to support the gallery of the Lyric Theater, Cleveland, Ohio, is shown in Fig. 337. This girder is



Longitudinal Sectional Elevation of Part of Building.

Fig. 336.—Longitudinal Section, Sub-station Building, Brooklyn Rapid Transit Co.



Section on Center Line.

Fig. 337.—Long Span Girder Carrying Gallery of Lyric Theatre, Cleveland, O.

55 ft. in clear span, 60 ins. deep and 12 ins. wide at the bottom and 20 ins. at the top. The reinforcement consists of round rods and stirrups, as shown. This girder was tested with a maximum

load of 44 tons, giving a total deflection of $\frac{19}{32}$ in. No evidences of cracks or other injuries were visible.

The general features of Unit system of construction are shown in Fig. 338. The details of the Unit girder frame were shown in Figs. 95 and 96. This system is being used in the construction of an eight-story manufacturing building in New York City. The building is approximately 125 × 137.5 ft. in plan. The wall columns are rectangular in section and reinforced with 4, 6 and 8 round rods. The interior columns for the four upper stories are of reinforced concrete of square section reinforced with 4 and 6 round rods. The lower interior columns are of cast iron and have the open section at the beam and column levels described on page 479, permitting the latter to run continuously through them. The first floor is designed for a live load of 200 lbs. per sq. ft.

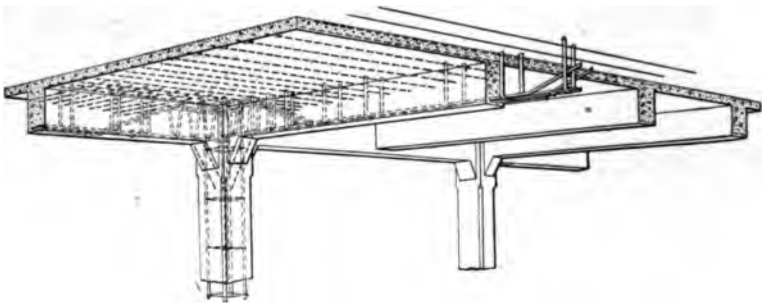


Fig. 338.—Typical Construction Unit Concrete Steel Co.'s System.

and the upper floors for 150 lbs. per sq. ft. The slab reinforcement consists of $\frac{7}{16}$ in. round rods, spaced 6 ins. centers, the slabs being 5½ and 4½ ins. thick. The columns are spaced 12 ft. centers longitudinally and 17 ft. 4 ins. transversely.

The building recently constructed for the Bush Terminal Company, located in South Brooklyn, N. Y., is an example of a reinforced concrete building of unusually heavy construction for warehouse and factory purposes. The reinforced concrete details were worked out and the construction supervised by Bertine & Son, under the direction of the then chief engineer of the Bush Terminal Company, Mr. E. P. Goodrich. This building is 600 ft. long, 75 ft. wide and has six stories and a basement. A pile foundation was necessary, as the soil in this locality is all made ground.

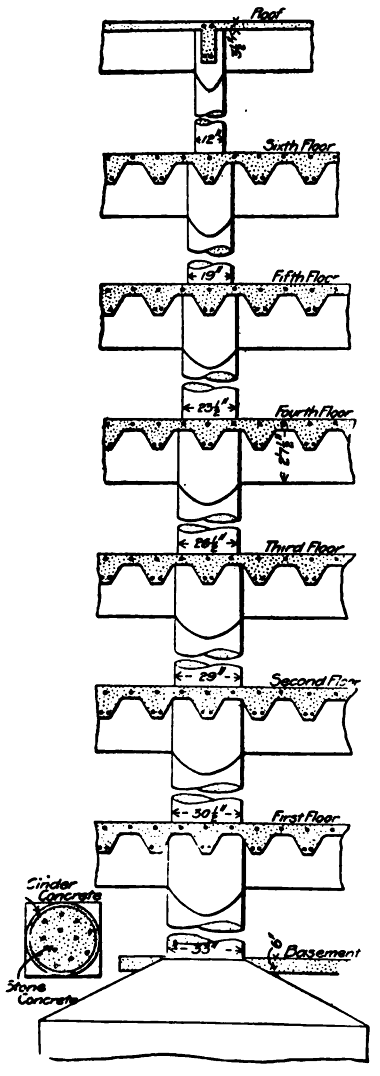


Fig. 339.—Column and Floor Construction. Bush Terminal Co.'s Factory.

Three rows of interior columns and two rows of wall columns extend the full length of the building. Longitudinal wall and floor girders span between the columns, which are spaced 16.5 ft. apart on centers. In a transverse direction one line of columns is placed on the center line of the building and one row 17 ft. 8 ins. on each side of it, leaving an outside space of 20 ft. between the last row of columns and the exterior of the wall columns or piers. The columns are connected longitudinally by massive reinforced concrete girders, which divide the floor space into two center longitudinal panels, 17 ft. 8 ins. wide on centers, and two outside panels 18 ft. wide in the clear. The panels extend from end to end of the building, a distance of 500 ft., exclusive of the somewhat narrower extension it has at the ends of the building, where there is a different arrangement of the columns and girders.

In the main portion of the building the floors in these longitudinal panels are carried entirely by the longitudinal girders, without any transverse beams or girders. The regular floor slabs are of special construction, designed to economically span the panel width. They have, as shown in the details, a continuous smooth horizontal upper surface 4½ ins. thick, strengthened by transverse ribs, 6 ins. deep and 22 ins. apart on centers, which give the ceilings a corrugated or trough-like appearance. These corrugations are staggered in adjacent panels, so that a cross-section through the center of a corrugation in one panel cuts through the center of a rib in the adjacent panels, and vice versa. The floors weigh about 100 lbs. per sq. ft. and are proportioned for a live load of 450 lbs. per sq. ft. (See Fig. 339 for sketch of floor.) The wall and center girders have rectangular cross-sections, and are reinforced with round bars wired together to form a rigid

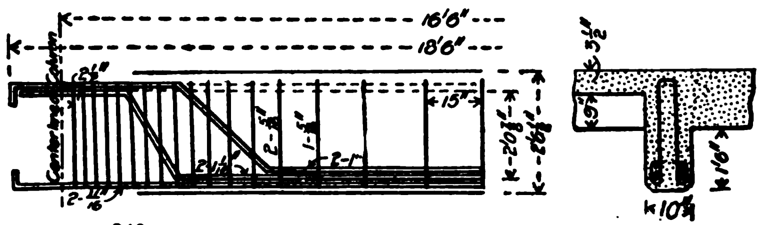


Fig. 340.—Intermediate Girders, Bush Terminal Co.'s Factory.

skeleton work of the Bertine system (see Figs. 340, 341). They have a width of 10 ins. and a depth to the floor slab of 30½ ins. Reinforcement consists of two 1-in. and two $1\frac{1}{16}$ in. round rods horizontal in the lower edge of the beam at the middle part, bent up at an angle to the top of the girder at both ends. In addition to this, there are two $\frac{11}{16}$-in. straight rods extending through the lower part of the girder from end to end. The rods are grouped in two sets of three each, vertically on each side of the

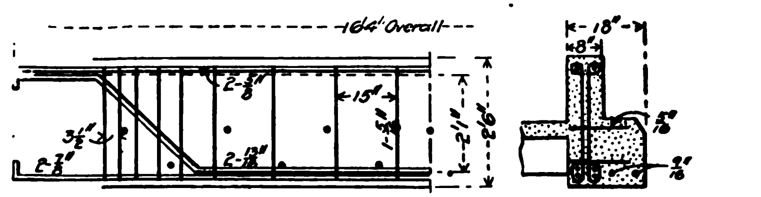


Fig. 341.—Wall Girder, Bush Terminal Co.'s Factory.

beam, and are connected by 37 vertical stirrups made of $\frac{5}{16}$-in. rods wired to them and spaced at variable distances, as determined by the diagram of shearing stresses in the beam.

The flat roof is proportioned for a uniform live load of 50 lbs. per sq. ft. and consists of a flat slab of reinforced concrete 3½ ins. thick, supported on the reinforced concrete longitudinal girders and on similar transverse girders at the columns and at intermediate points. The interior columns are of hooped construction, and are fully described on page 477. The exterior columns are of rectangular section. The exterior walls are of brick work supported at each story by reinforced concrete girders, which, with the interior girders, are made integral with the floor slabs

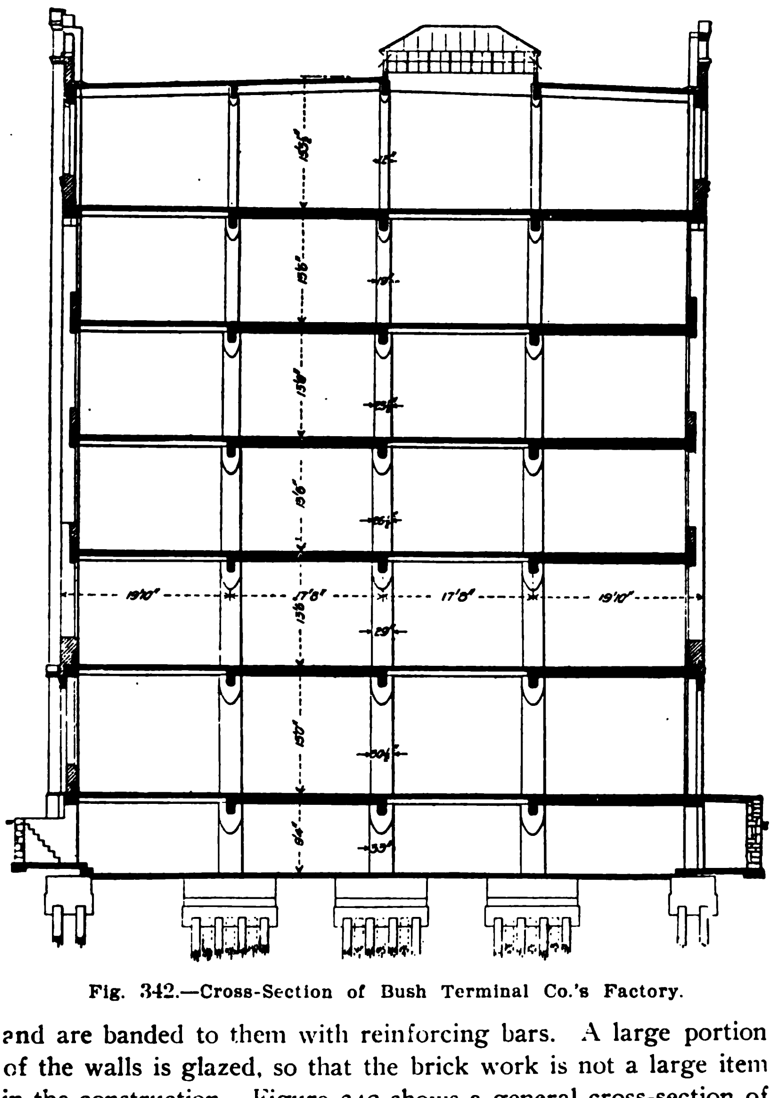


Fig. 342.—Cross-Section of Bush Terminal Co.'s Factory.

and are banded to them with reinforcing bars. A large portion of the walls is glazed, so that the brick work is not a large item in the construction. Figure 342 shows a general cross-section of the building.

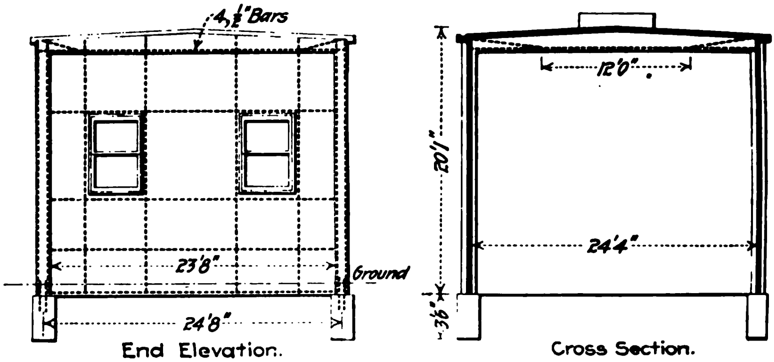


Fig. 343.

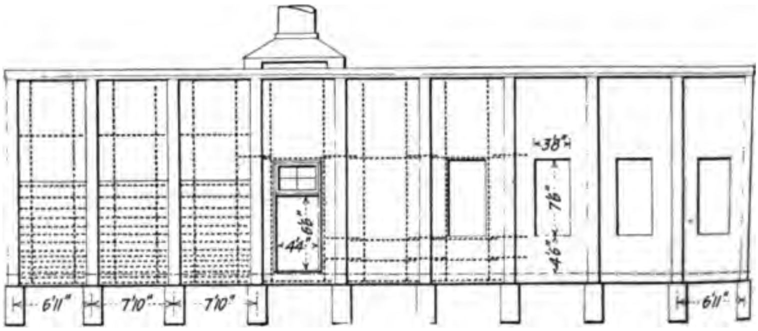


Fig. 344

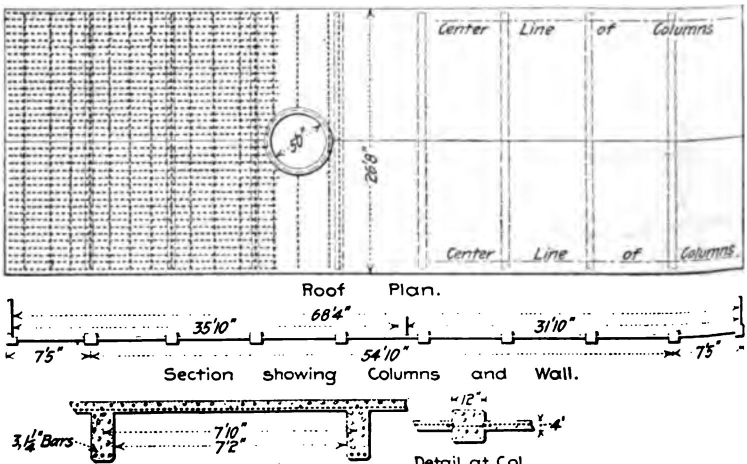


Fig. 345.

Figures 343 to 345 show plans of a one-story power house for the Ridgefield Electric Co., Ridgefield, Conn., designed by Mr. E. S. Ball, Assoc. M. Am. Soc. C. E., of New York. The building is 70 x 25 ft. in plan, and 20 ft. high. The construction consists of 12 x 12-in. wall columns, spaced 7 ft. 10 in. centers, carrying 8-in. cross girders 22 ins. deep at the center and 16 ins. at the ends, reinforced by three 1¼-in. bars. The cross girders carry the 4-in. roof slab, reinforced by ½-in. rods, spaced 8-in. centers, and running from girder to girder, forming a continuous roof slab. The wall columns are reinforced with four ½-in. vertical rods. The wall slabs were built in place after the construction of the wall columns and roof girders, and are reinforced with rods, as shown. In the end of the building, where the boiler is located, the side wall is 6 ins. thick and reinforced as shown, to act as a storage bin for coal. The general features of the building will be understood from the drawings. This building is a good illustration of a type of building of moderate size to which reinforced concrete may with economy be applied. It gives a fireproof structure at a moderate cost.

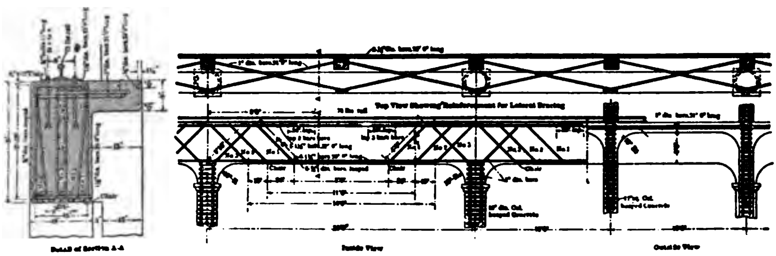


Fig. 346.—Crane Run Girder, Taylor-Wilson Mfg. Co.'s Shops.

A good example of a reinforced concrete building used to replace the type of building so commonly used for what is generally called "mill building construction," is that used for the shop at McKees Rocks, Pa., for the Taylor-Wilson Manufacturing Co. This structure was designed and built by Mr. Robert A. Cummings, M. Am. Soc. C. E., of Pittsburg, Pa. The column and girder construction is of the Cummings system, described on pages 247 and 474. Figure 346 shows the general system of reinforcement for the crane-run girders. A cross-section of the main girder is shown in Section A-A. A cross-section of the shop is shown in Fig. 347. As will be seen, the main features of the building consist of Cummings columns, the outside column being 12 ins. square and about 16 ft. high. These support the walls of the lean-to. Two rows of interior columns 20 ins. in diameter and the side columns carrying the crane-run girders support the roof. These columns are reinforced with four vertical rods to which are attached a series of hoops $1\frac{1}{2}$ ins. wide and $\frac{1}{8}$ in. thick, spaced 4 ins. apart. The main girders of the building were designed for a 30-ton crane, and span 20 ft. between columns. A section of this beam is 18 x 36 ins. and has an upper flange, reinforced as shown in the figure, to take care of the transverse thrust of the crane. The roof construction is a good example of the application of reinforced concrete arch slab. This concrete

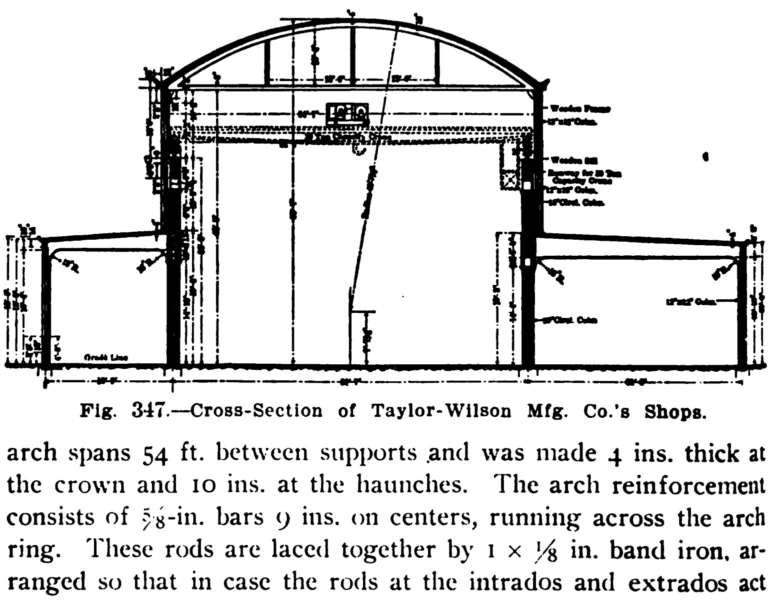


Fig. 347.—Cross-Section of Taylor-Wilson Mfg. Co.'s Shops.

arch spans 54 ft. between supports and was made 4 ins. thick at the crown and 10 ins. at the haunches. The arch reinforcement consists of $\frac{5}{8}$-in. bars 9 ins. on centers, running across the arch ring. These rods are laced together by 1 x $\frac{1}{8}$ in. band iron, arranged so that in case the rods at the intrados and extrados act in compression, there will be no danger of buckling.

The thrust of the arch is taken up by tie rods made up of two 3 x $2\frac{1}{2}$ x $\frac{7}{16}$-in. angle irons, spaced 10 ft. apart. These tie rods are bolted to two 10-in., 15-lb. channels, placed back to back, as shown in Fig. 348. The channels distribute the load due to the thrust between the rods. A light skewback casting was placed in the upper channel to act as a spacing bar for the roof rods and to transfer the thrust of the arch through the medium of the channels to the tie rods. Expansion joints were made every 10

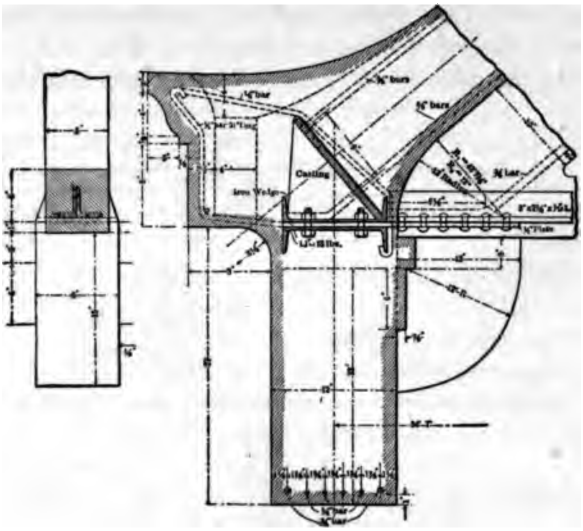


Fig. 348.—Tie Rod Construction for Arched Roof, Taylor-Wilson Mfg. Co.'s Shops.

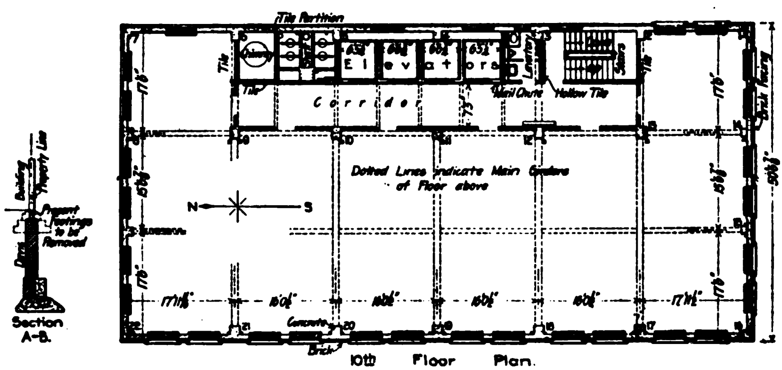


Fig. 349.—Plan of Tenth Floor of Ingalls Building.

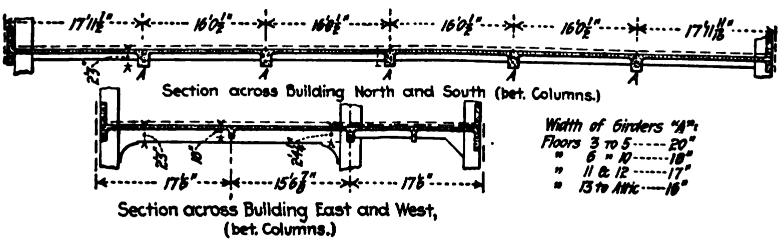


Fig. 350.—Sections of Tenth Floor of Ingalls Building.

ft. in the arch to care for the large temperature stress that would naturally be developed in such a large thin area.

Probably the most pretentious reinforced concrete building in

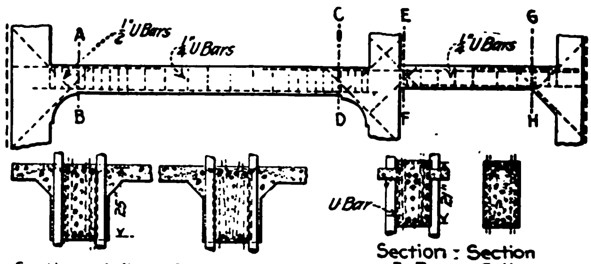


Fig. 351.—Typical Girder Construction, Ingalls Building.

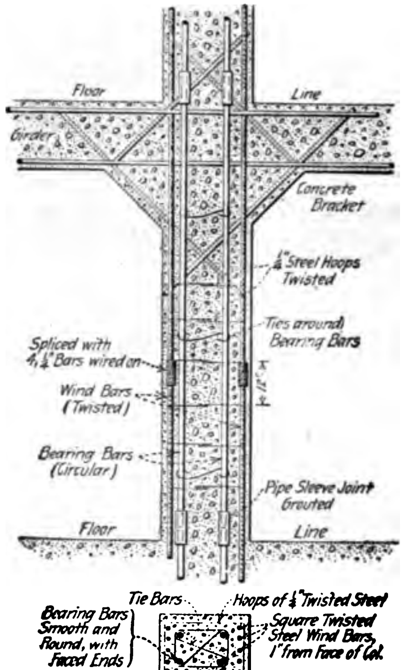


Fig. 352.—Girder and Column Connection, Ingalls Building.

this country is the 16-story Ingalls office building at Cincinnati, Ohio. This building is of Ransome construction. This structure is 213 ft. high from the sidewalk to the cornice, and is 100 x 50½ ft. in plan. A row of interior columns running lengthwise of the building divide it transversely into two floor bays, one 17.5 ft. and the other 33 ft. wide. Figure 349 is a plan of the tenth floor, and shows the location of columns and main girders, while Fig. 350 shows both longitudinal and transverse sections of the floor. The main girders are 36 ins. deep on the first floor, 34 ins. on the second, and 27 ins. on the floors above, and vary in width from 16 to 20 ins. The depth of the girders includes the thickness of the floor slab, which is 7 ins. on the first floor and 5 ins. on the floors above.

The girders are so designed that they are fixed at the walls and columns. The typical girder construction is shown in Fig. 351; they have reinforcing rods and stirrups at both top and bottom. Inclined bars are used to stiffen the column and girder connections to care for bending at the columns due to wind pressure. The connections at the columns are clearly shown in Fig. 352. The following list gives the number and size of reinforcing rods used in the various types of girders:

Mark.	Bars, No.	Size, ins.	Length, ft.	Mark.	Bars, No.	Size, ins.	Length, ft.
1	4	1¼	10	11	2	¾	6
2	2	1	13	12	2	1¼	34
3	2	1	9¾	13	2	1¼	14
4	2	1	34	14	2	1¼	11¼
5	2	1	34	15	2	¾	5
6	2	½	7	16	2	¾	8
7	2	1	8¼	17	2	¾	7
8	2	1¼	10	18	2	1	13
9	2	1¼	11½	19	2	1	16
10	2	¾	5½	..	..	...	..

U BARS IN LEFT-HAND SPAN.

4 Type X, ¼-in., 10 Type X, ½-in............5 ft. 4 ins. long.
14 Type Y, ¼-in...........................7 ft. 8 ins. long.

U BARS IN RIGHT-HAND SPAN.

10 bars, ¼-in...............................4 ft. 4½ ins. long.

The floors are 7 ins. thick on the first floor and 5 ins. on all others, and are reinforced with a network of two layers of ¾-in. twisted rods laid at right angles and spaced from 12 to 16 ins. center. The floors are divided by the longitudinal and transverse girders into panels 16 ft. square, and are figured as a flat floor slab supported on four edges. The floors are calculated for a live load of 200 lbs. on the first floor and 80 lbs. on the second floor and 60 lbs. per sq. ft. on all others. The concrete used was made of 1 part Lehigh Portland cement, 2 parts sharp sand and 4 parts 1-in. crushed limestone or gravel. Both the stone and gravel were crushed and used without screening.

Plain Arch Floors.—Reinforced concrete arches may be used in the construction of monolithic floors of wide spans. The thrust of the arch on the walls and the weight of the spandrel filling largely modify the form and rise of arches in buildings. The

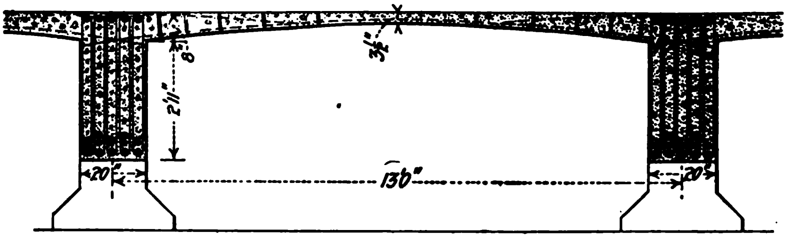


Fig. 353.—Arch Slab Floor Carried by Arch Ribs.

great weight of the spandrel filling for arches of even moderate rise make the use of a flat arch imperative. A flat extrados with a horizontal reinforcement near its upper surface in addition to the usual intradosal reinforcement is the most common form. The upper reinforcement is supposed to modify the thrust, acting along the theoretical axis of the arch. This type of arch is an

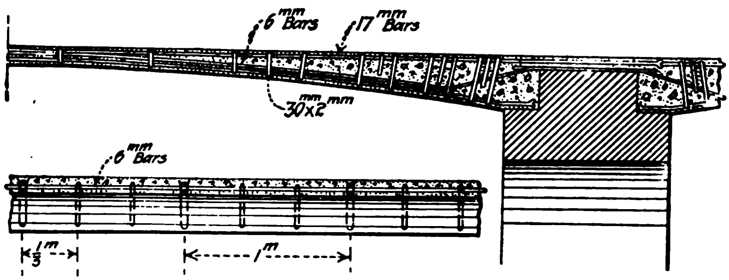


Fig. 354.—Plain Arch Floor, Petit Palais des Beaux Arts, Paris Exposition of 1900.

economical form of construction for heavily loaded or wide span floors.

Fig. 353 shows an arch floor of Hennebique construction with a span of 13 ft., supported by 50-ft. arch rib girders. The thickness of the arch is 3½ ins. at the crown and 8 ins. at the haunches. The dotted lines show the location of the reinforcement in the slab.

The Wünch arch, as applied to monolithic floor construction, is entirely analogous to that shown in Fig. 490, the only difference being in the end attachment. The rise of the arch is usually $\frac{1}{10}$ of the span. The two reinforcements are rigidly attached at the ends to an anchoring rod placed vertically in the side wall masonry. The Wünch arch has been more extensively employed in bridge construction than in buildings.

Figure 354 shows the section of an arch floor constructed in the Petit Palais des Beaux Arts at the Paris Exposition of 1900. A gallery of 6.10 meters (20 ft.) was spanned by this arch. The thickness at the crown was $3\frac{1}{8}$ ins. and $8\frac{5}{8}$ ins. at the springing

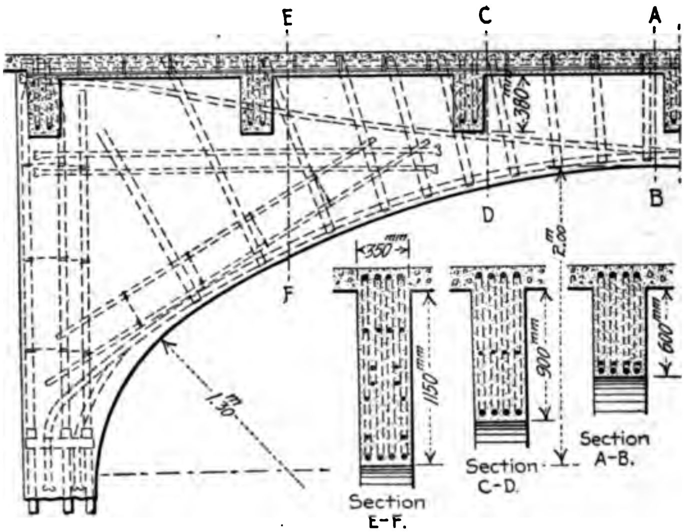


Fig. 355.—Ribbed Arch Floor for Machine Shops at Nantes, France.

points. These arches were designed to carry a combined dead and live load of 220 lbs. per sq. ft.

Ribbed Arch Floors.—Ribbed arches supporting a flat floor plate are more common than plain arch floors. Sometimes plain arches are used to span between arch ribs. Figure 355 shows a good example of a flat floor supported by arch ribs used by Hennebique in the construction of a machine shop at Nantes, France. The clear arch span is 26 ft., with a rise of 6.5 ft.

Figure 356 shows a longitudinal and transverse section of an arch floor, supported by arch ribs, used over a gallery in the Petite Palais des Beaux Arts at Paris. The ribs had a span of

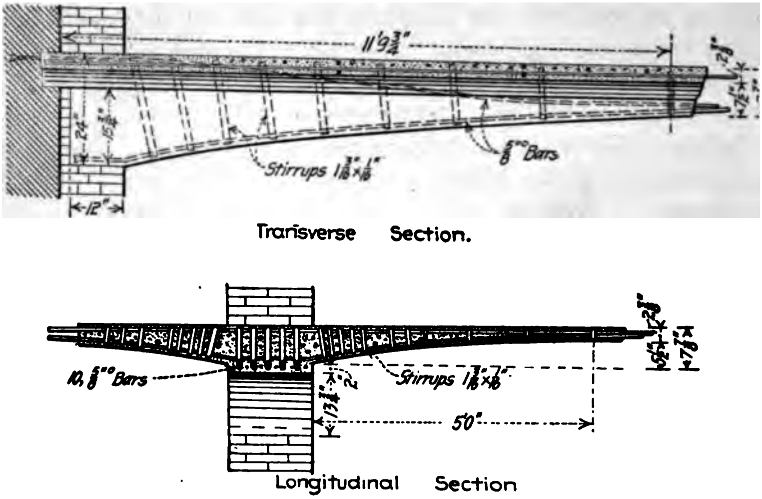


Fig. 356.—Ribbed Arch Floor, Petit Palais des Beaux Arts, Paris Exposition of 1900.

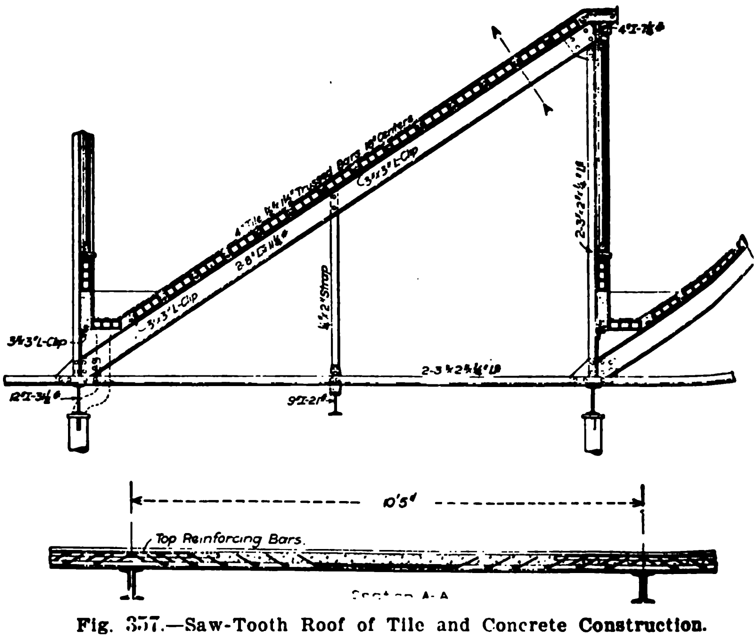


Fig. 357.—Saw-Tooth Roof of Tile and Concrete Construction.

7.2 meters (27.6 ft.) and are spaced 3.48 meters (11.4 ft.) centers, the latter distance being the span of the plain arches forming the floor.

The ribs were 10 ins. thick at the crown and 24 ins. at the spring. The arch floor slabs were $2\frac{5}{16}$ ins. thick at the crown

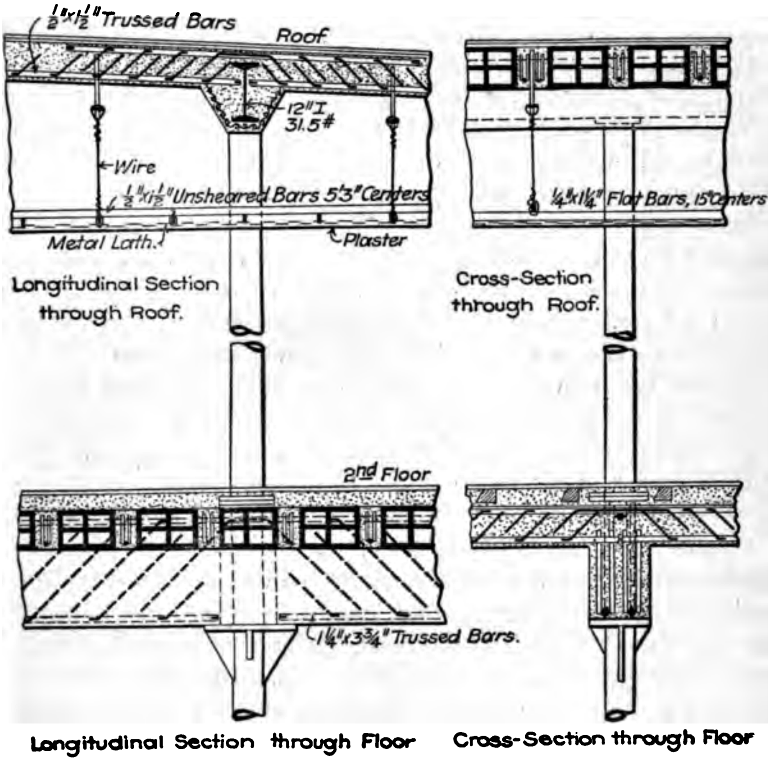


Fig. 358.—Floor Construction of Tile and Concrete.

and 8 ins. at the spring. This floor system was designed to carry a floor load of 220 lbs. per sq. ft.

Tile-Concrete Construction.—A type of roof and floor construction which has been extensively employed for building by one well-known company is shown in Figs. 357 and 358. It consists of a ribbed slab having the space between the ribs filled with hollow tile, the lower part of the tile being flush with the bottom of the beam. The object of the use of the tile is to simplify the construction of the forms and to secure a flat ceiling. The ribs are reinforced with Kahn bars in the usual manner.

In this construction the hollow tile are laid end to end, on wood lagging, in straight rows, the rows of tile being about 4 ins. apart. In these spaces are dropped first 1 in. of concrete, then a Kahn trussed bar with the diagonals pointing upward, and the spaces are filled with a wet mixture of cement concrete, thus forming a series of reinforced concrete beams approximately 16 ins. on centers. The depth of the tile and depth of the tile-concrete floor slab varies with the span, load to be carried, etc. The construction shown in Figs. 357 and 358 was used for the roof and floor of a factory for the American Arithmometer Co., Detroit, Mich. Figure 357 shows the manner of using concrete-tile roof slab carried by steel trusses for a saw-toothed roof, while Fig. 358 shows details of both roof and floor construction.

In the construction of slabs of this type some positive means should be employed to hold the tile rigidly in position until the concrete has been put in and set. The author has seen tile displaced as much as 2 or 3 ins. during progress of construction. The strength of a floor rib would appear to be an uncertain quantity if even a single tile is displaced to any extent.

The concrete slab covering used over the tile is at times as thin as 2 ins. This would appear to be bad practice for the lean mixtures and large size stone commonly used in this country. Another structural weakness is the lack of any transverse rods such as are usually considered necessary to prevent shrinkage cracks and strengthen the floor transversely. An advantage of this type of floor is the air space in the tile, which serves to break up sound passing through the floor. It is claimed that the concrete dries out much more quickly when tile is used than in a solid concrete floor. This should permit the construction to progress much more rapidly, and if the work is properly done should give satisfaction.

Continuous Beams and Slabs.—Considerable economy of material results when beams and girders are considered as fixed at two or more supports. It is not considered good practice to apply the principles of continuity to the design of beams and girders, but in certain cases they may be applied to the design of floor slabs. When slabs and beams are treated as continuous in design, much better workmanship must be obtained than is customary, or local failures may occur and the whole structure fail. When moving loads or vibrations of machinery will come upon a structure, it is better practice to consider the floor slabs, beams and girders as freely supported, thereby securing a higher factor of safety. The possibility of future changes in the floor openings, on account of the exigencies of business, also make continuous floor slabs undesirable.

Let w represent the load per lineal foot and l the span in feet. Then the bending moment at the center of a beam uniformly loaded and freely supported on two edges is $\frac{1}{8}$ w l^2, and when it is fixed at the supports the bending moment is $M = \frac{1}{24}$ w l^2. On account of the deflection the beam will not be absolutely fixed at the supports, and many designers use the formula $M = \frac{1}{10}$ w l^2. For square slabs freely supported on all four edges $M = \frac{1}{16}$ w l^2, and for square slabs fixed at all four edges the bending moment $M = \frac{1}{24}$ w l^2. On account of the slab not being truly fixed, it is usual to take $M = \frac{1}{20}$ w l^2. When rectangular slabs are used, the bending moment varies with the ratio between the transverse and longitudinal span. If L represents the longer span, and l the shorter, the bending moments, as given by the above formulas, may be obtained by multiplying by the coefficient $\frac{l^4}{l^4 + L^4}$ when the shorter span l is being considered, and by $\frac{L^4}{L^4 + l^4}$ when the longer span L is being considered. The greatest bending moment will be for the shorter span.

Thus, for the shorter span, the coefficient becomes, when L is infinite, $\frac{L^4}{L^4 + l^4} = 1.$

When $L = l$ $\qquad \frac{L^4}{L^4 + l^4} = \frac{1}{2}.$

When $L = 2\,l$, $\qquad \frac{L^4}{L^4 + l^4} = 0.94.$

For the longer span the coefficient becomes when L is infinite,

$\frac{l^4}{l^4 + L^4} = 0.$

When $L = l$, $\qquad \frac{l^4}{l^4 + L^4} = \frac{1}{2}.$

When $L = 2\,l$ $\qquad \frac{l^4}{l^4 + L^4} = .06.$

The above coefficients should be used for computing the bending moment when the slabs are to be reinforced in both directions, it being remembered that the proper formula for the bending moment under the given condition is to be multiplied by the coefficient for long or short span, as the case may be. It should also be remembered that slabs considered as fixed should have reinforcement near its top face, over the supporting beam or girder.

New York Building Regulations.—The rules under which reinforced concrete structures are erected in New York City are given in the following regulations:

1. The term "concrete-steel" in these Regulations shall be understood to mean an approved concrete mixture reinforced by steel of any shape, so combined that the steel will take up the tensional stresses and assist in the resistance to shear.

2. Concrete-steel construction will be approved only for buildings which are not required to be fireproof by the Building Code, unless satisfactory fire and water tests shall have been made under the supervision of this Bureau. Such tests shall be made in accordance with the Regulations fixed by this Bureau and conducted as nearly as practicable in the same manner as prescribed for fireproof floor fillings in Section 106 of the Building Code. Each company offering a system of concrete-steel construction for fireproof buildings must submit such construction to a fire and water test.

3. Before permission to erect any concrete-steel structure is issued, complete drawings and specifications must be filed with the Superintendent of Buildings, showing all details of the construction, the size and position of all reinforcing rods, stirrups, etc., and giving the composition of the concrete.

4. The execution of work shall be confided to workmen who shall be under the control of a competent foreman or superintendent.

5. The concrete must be mixed in the proportions of one of cement, two of sand and four of stone or gravel; or the proportions may be such that the resistance of the concrete to crushing shall not be less than 2,000 lbs. per square inch after hardening for twenty-eight days. The tests to determine this value must be made under the direction of the Superintendent of Buildings.

The concrete used in concrete-steel construction must be what is usually known as a "wet" mixture.

6. Only high-grade Portland cements shall be permitted in concrete-steel construction. Such cements, when tested neat, shall, after one day in air, develop a tensile strength of at least 300 lbs. per square inch; and after one day in air and six days in water shall develop a tensile strength of at least 500 lbs. per square inch; and after one day in air and twenty-seven days in water shall develop a tensile strength of at least 600 lbs. per square inch. Other tests, as to fineness, constancy of volume, etc., made in accordance with the standard method prescribed by the American Society of Civil Engineers' Committee may, from time to time, be prescribed by the Superintendent of Buildings.

7. The sand to be used must be clean, sharp grit sand, free from loam or dirt, and shall not be finer than the standard sample of the Bureau of Buildings.

8. The stone used in the concrete shall be a clean, broken trap rock, or gravel, of a size that will pass through a three-quarter inch ring. In case it is desired to use any other material or other kind of stone than that specified, samples of same must first be submitted to and approved by the Superintendent of Buildings.

9. The steel shall meet the requirements of Section 21 of the Building Code.

10. Concrete steel shall be so designed that the stresses in the concrete and the steel shall not exceed the following limits:

Extreme fibre stress on concrete in compression....	500	lbs. per sq. in.
Shearing stress in concrete.........................	50	"
Concrete in direct compression.....................	350	"
Tensile stress in steel............................	16,000	"
Shearing stress in steel...........................	10,000	"

11. The adhesion of concrete to steel shall be assumed to be not greater than the shearing strength of the concrete.

12. The ratio of the moduli of elasticity of concrete and steel shall be taken as 1 to 12.

13. The following assumption shall guide in the determination of the bending moments due to the external forces. Beams and girders shall be considered as simply supported at the ends, no allowance being made for continuous construction over supports. Floor plates, when constructed continuous and when provided with reinforcement at top of plate over the supports, may be

treated as continuous beams, the bending moment for uniformly distributed loads being taken at not less than $\frac{WL}{10}$; the bending moment may be taken at $\frac{WL}{20}$ in the case of square floor plates which are reinforced in both directions and supported on all sides. The floor plate, to the extent of not more than ten times the width of any beam or girder, may be taken as part of that beam or girder in computing its moment of resistance.

14. The moment of resistance of any concrete-steel construction under transverse loads shall be determined by formulæ based on the following assumptions:

(*a.*) The bond between the concrete and steel is sufficient to make the two materials act together as a homogeneous solid.

(*b.*) The strain in any fibre is directly proportionate to the distance of that fibre from the neutral axis.

(*c.*) The modulus of elasticity of the concrete remains constant within the limits of the working stresses fixed in these Regulations.

From these assumptions it follows that the stress in any fibre is directly proportionate to the distance of that fibre from the neutral axis.

The tensile strength of the concrete shall not be considered.

15. When the shearing stresses developed in any part of a concrete-steel construction exceed the safe working strength of concrete, as fixed in these Regulations, a sufficient amount of steel shall be introduced in such a position that the deficiency in the resistance to shear is overcome.

16. When the safe limit of adhesion between the concrete and steel is exceeded, some provision must be made for transmitting the strength of the steel to the concrete.

17. Concrete-steel may be used for columns in which the ratio of length to least side or diameter does not exceed twelve. The reinforcing rods must be tied together at intervals of not more than the least side or diameter of the column.

18. The contractor must be prepared to make load tests on any portion of a concrete-steel construction, within a reasonable time after erection, as often as may be required by the Superintendent of Buildings. The tests must show that the construction will

sustain a load of three times that for which it is designed without any sign of failure.

Approved September 9th, 1903.

Walls and Partitions.—Reinforced concrete walls and partitions are used both in steel frame and reinforced concrete buildings. They may be self-sustaining, be supported by a steel framework or by reinforced concrete column and beam. Usually, however, these walls have only to sustain their own weight throughout the height of one story and for a single panel width between columns. Ordinarily for outside walls the only pressure brought upon them is that due to wind, which seldom exceeds 30 lbs. per sq. ft. Walls can therefore be made very light.

It is, however, advisable, when side walls are used, to make them 3½ or 4 ins. in thickness and reinforce them with rods running in both directions. If this is not done there is danger of the walls cracking, due to changes in temperature, and also it is difficult to construct a satisfactory wall less than 3½ to 4 ins. in thickness.

The reinforcement generally consists of ¼ to ½ in. vertical rods, spaced from 1 ft. 6 ins. to 3 ft. apart, and horizontal rods of the same size, spaced from 1 to 2 ft. apart.

In warehouses, factories, etc., considerable lateral pressure is at times brought upon both partitions and outside walls. Under these circumstances the wall slab must be figured as a floor slab for a safe unit pressure and properly proportioned for the span.

In buildings where a large percentage of the wall is used for windows, the wall proper consists of little more than reinforced columns with deep wall beams forming belt courses between them and acting as lintels and window bases. These wall girders support the floor between columns and have reinforcing rods near the top and bottom.

When any additional filling is needed between columns it may consists of a reinforced concrete slab, cement blocks, brick or terra cotta.

Walls and partitions may be either solid or hollow. Solid walls, when used for exterior walls, are not very satisfactory, as, owing to the higher temperature indoors, moisture condenses on their inside face, making the rooms damp and unhealthy.

Little trouble of this kind is experienced when double walls are used. These consist of two thin slabs reinforced with vertical and horizontal rods, or some form of Monier netting, and connected at intervals by cross ribs of concrete. While double walls are, on the whole, very satisfactory, they have not been extensively used owing to their high cost. In places where solid walls are not objectionable their small thickness makes a great saving in floor space.

In cities where land values are very high this fact deserves

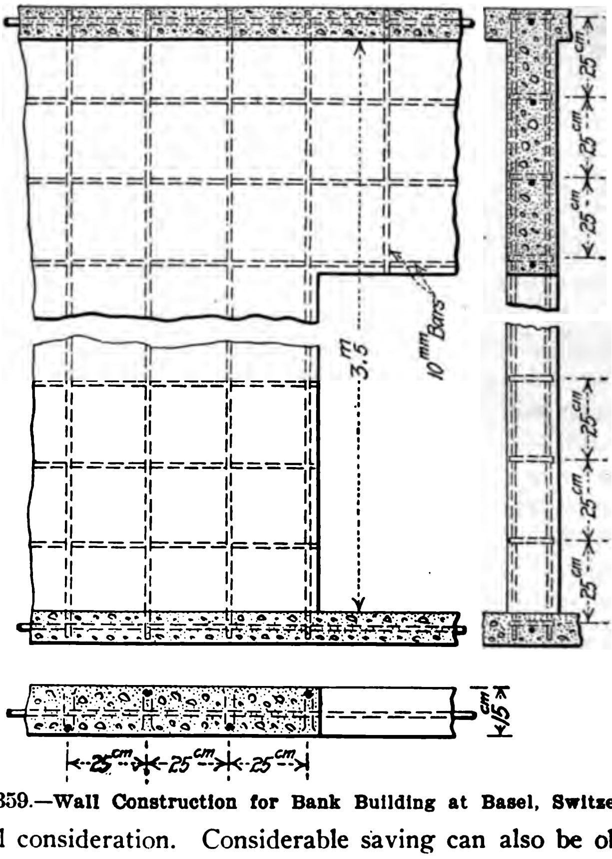


Fig. 359.—Wall Construction for Bank Building at Basel, Switzerland.

careful consideration. Considerable saving can also be obtained by their use as partitions in place of brick or other materials.

The requirements of the building laws of various cities, and the requirements of the Board of Fire Underwriters, requiring brick walls, or concrete walls of the same thickness as brick walls, prevent the extensive use of reinforced concrete for exterior walls in cities.

Concrete walls, either double or solid, when used for exterior walls, are often veneered with terra cotta, brick, stone or marble.

In the Hennebique system of wall construction, vertical rods are placed near each face, and horizontal rods along the middle of the slab. Figure 160 shows the character of this form of reinforcement. Christophe, in "Le Beton Armé," gives an example of a monolithic wall of Hennebique construction used in a bank

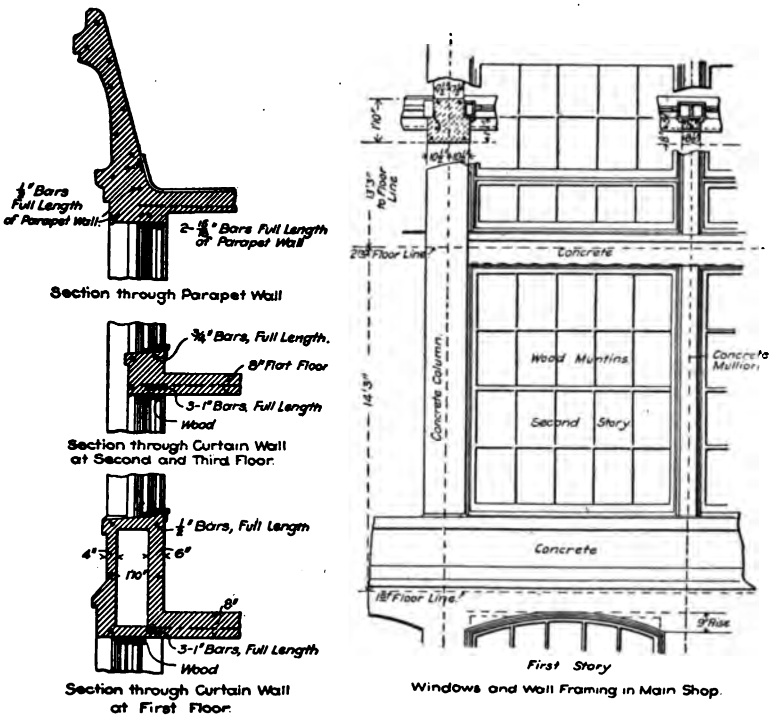


Fig. 360.—Window and Wall Framing for Main Shop, United Shoe Machinery Co.

building in Basel, Switzerland. Fig. 359 shows a section of this wall, including a doorway.

Stirrups were used on the vertical rods only. The wall was designed to resist a lateral thrust of 50 lbs. per sq. ft.

The standard partitions used in the United Shoe Machinery Co.'s building are 4 ins. in thickness, reinforced with ¼-in. vertical rods, spaced 3 ft. centers, and placed ½ in. from alternate faces of the wall; and ¼ in. horizontal rods, spaced 12 in. centers.

The horizontal rods are wired to the inner faces of the vertical rods and are also located near alternate faces of the partition. Two ¼-in. vertical bars are placed on each side of all openings and one ¼-in. bar at each corner, and one ½-in. bar over each doorway. The vertical bars project into the floor and ceiling and

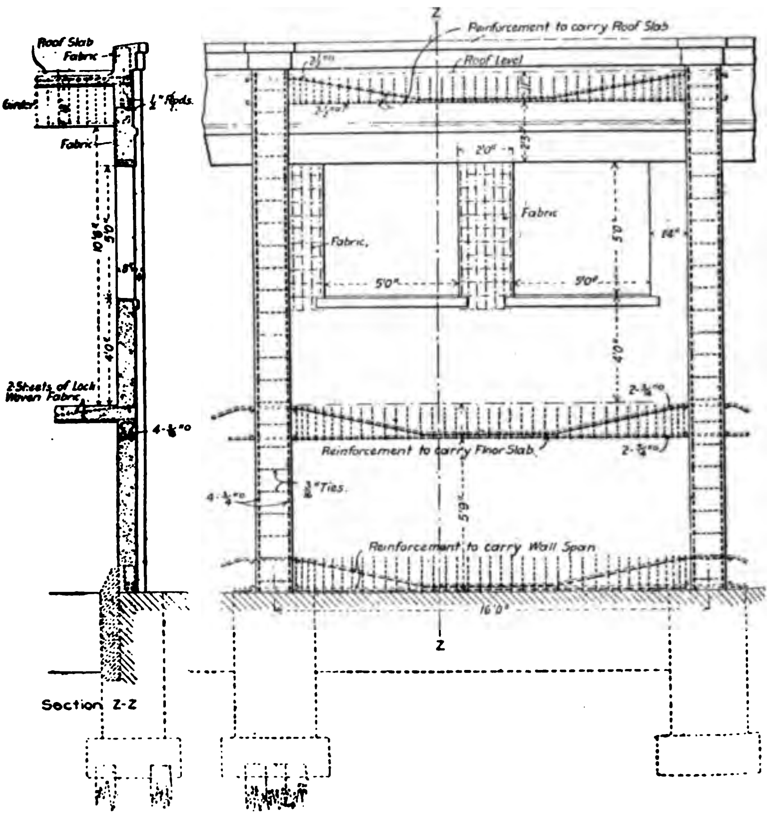


Fig. 361.—Wall Construction, Central Felt and Paper Co.'s Factory.

anchor the partitions securely to them. Figure 360 shows window and wall framing for main shop.

The outer walls of these buildings are 70 per cent. window space, and consist almost entirely of wall columns, girders or belt courses and cornice, so that above the sills of the first-floor windows there is practically no wall surface proper. The character of the wall surfaces, belt courses and cornice is shown in Fig. 360.

Figure 361 shows wall construction used for the Central Felt and Paper Co.'s building, Long Island City.

The above details show the usual type of wall construction used in this class of buildings. When broader belt courses and vertical wall slabs are employed with a smaller percentage of window area, it is customary to use both horizontal and vertical rod reinforcement, and place one or more rods near the edge of the slab about all openings. Sometimes electrically welded wire or lock woven fabric is used for wall reinforcement. This material is especially adapted to the construction of thin curtain walls. At the edge of all openings it is customary to bend the fabric back into a U shape to strengthen the concrete about the opening.

Hollow concrete walls were used in the construction of the Pacific Coast Borax Co. building, Bayonne, N. J. The thickness of the two faces was 3½ ins., with an air space of 9 ins. in the first story. Both the air space and the walls decrease in thickness in the top stories. The reinforcement used is similar to that employed in the United States Shoe Machinery Co.'s building.

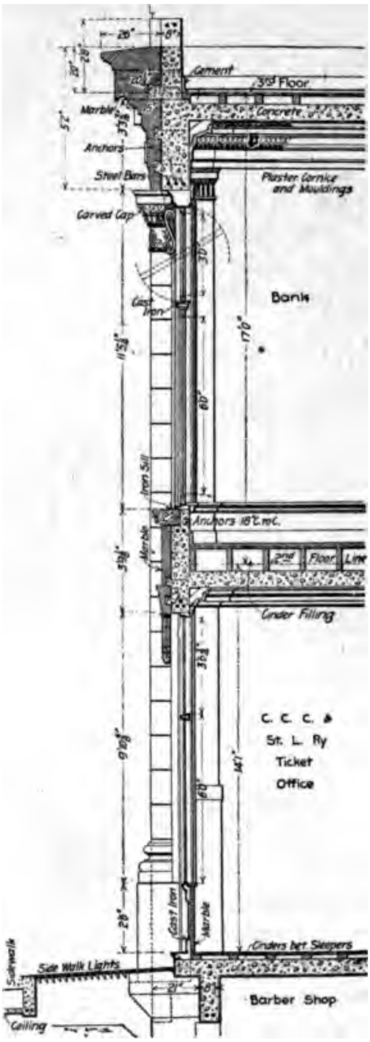


Fig. 362.—Section of Wall for Ingalls Building.

In the Ingalls building the walls are of concrete, 8 ins. thick, faced with marble in the lower stories and terra cotta in the upper stories. The concrete at the floor lines and between the top of the windows in the story below and the bottom of the one above consists of deep reinforced concrete girders. The reinforcement consists of two ½-in. rods placed 2 ins. above the top of the windows in the story below, and similar rods below the window sill of the story above. The remainder of the wall was reinforced with ¼-in. horizontal rods spaced 2 ft. apart. Vertical reinforcing rods are also placed 2 ins. from each window opening.

The marble and terra cotta facing is supported by projections in the concrete fitting into openings in the facing, forming a sort of dovetailing. Iron anchors tie the two together.

Figures 362 and 363 show the details of construction.

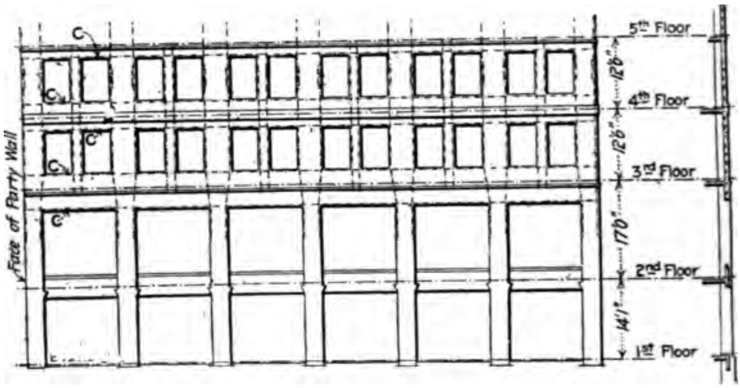


Fig. 363.—Elevation of Wall for Ingalls Building.

Concrete Wall Construction Without Wooden Forms.—The Weiderholt system of construction is one in which no wooden or temporary forms are required. By the use of hollow tile blocks of special shape, a thin shell of fire clay or cement is used as the mould and forms the finished exterior surface. The tile blocks are H-shaped in plan, the two long sides forming the inner and outer faces of the wall, while the web is reduced as much as possible and only enough material retained to hold the sides together while the concrete is being placed and tamped. By cutting away a portion of the web, space is secured for the horizontal reinforcing bars. By reducing the web as much as possible, practically a monolithic wall is secured. The vertical reinforcing bars are embedded in the foundation in the usual way and the tiles are laid between them, with horizontal bars at suitable intervals, after which the concrete is placed, the tiling and concreting being carried up as the wall progresses. This system is adapted to the construction of walls of buildings, grain and storage bins, chimneys, etc. When used for chimneys a hard tile may be used, which will protect the concrete from the action of flame from the furnace.

This type of construction is to be used for the smokestack of the Martin-Shaughnessy Warehouse Building, St. Louis, Mo. The smokestack will be 91 ft. high; for 50 ft. it will be 3 ft. in diameter inside, with walls 8 ins. thick; in the upper part the thickness of the walls will be reduced to 6 ins., giving an inside diameter of 3 ft. 4 ins. The reinforcement consists of ¾-in. square vertical bars, spaced 12 ins. centers, and ½-in. square horizontal bars, 12 ins. centers in the lower section and 24 ins. apart in the upper section. A 1 Portland cement to 3 parts clean river sand will be used for the concrete. Fig. 364 shows the form

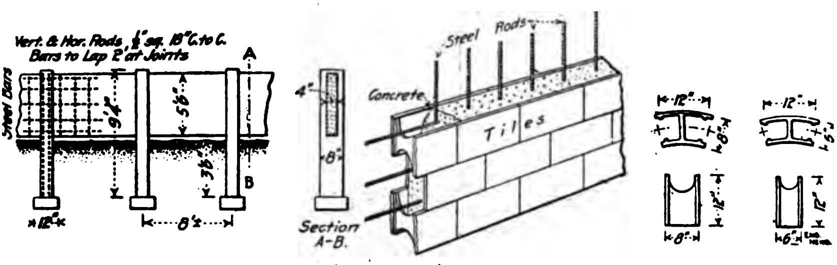


Fig. 364.—Wall Construction by the Weiderholt System.

of the tile used for walls and chimneys. This system is being introduced by the Atlas Construction Co., St. Louis, Mo.

Plaster Partitions.—A form of partition extensively used in fireproof hotel and office building construction consists of a single or double mesh of wire netting or expanded metal lath, supported by posts of wood or metal fastened to the framing of the floor or ceiling of a steel building or bedded in the concrete of a reinforced concrete building and plastered over on both sides with an inch or more of mortar. While timber studding has been used in this type of wall, metal is most frequently employed. Small channels or round rods, spaced from 12 to 16 ins. centers, depending upon the height of the wall, are used for studding. The wire mesh or expanded metal is placed on both sides of the uprights and wired to them, leaving an air space between when a double wall is desired, or expanded metal lath is woven between the upright rods of a wall of single thickness. Both sides of the partition are then covered with plaster. The Roebling partition, shown in Figs. 365 and 366, are good examples of this type of partition. As will be seen, one of these is a solid partition, while the other is double, with an air space.

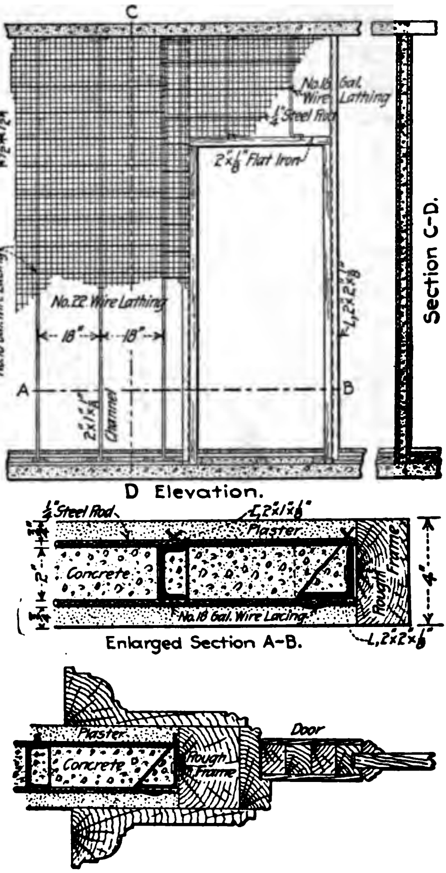


Fig. 365.—Solid Wall Construction, Roebling Type.

At all openings timber or channel iron casings are provided, to which the mesh is firmly fastened. In Europe a slight modification of this wall is used, a single Monier netting, with the carrying rods horizontal and the distribution rods vertical, being employed. When a thick wall is desired two slabs are built, with an air space between them.

Walls constructed of cast slabs, strengthened with some form of Monier netting, have been used in a few buildings in Europe, but, as far as the writer knows, have never been employed in this country. Hollow blocks, strung on vertical rods, is another form of wall construction sometimes employed in Europe. In America

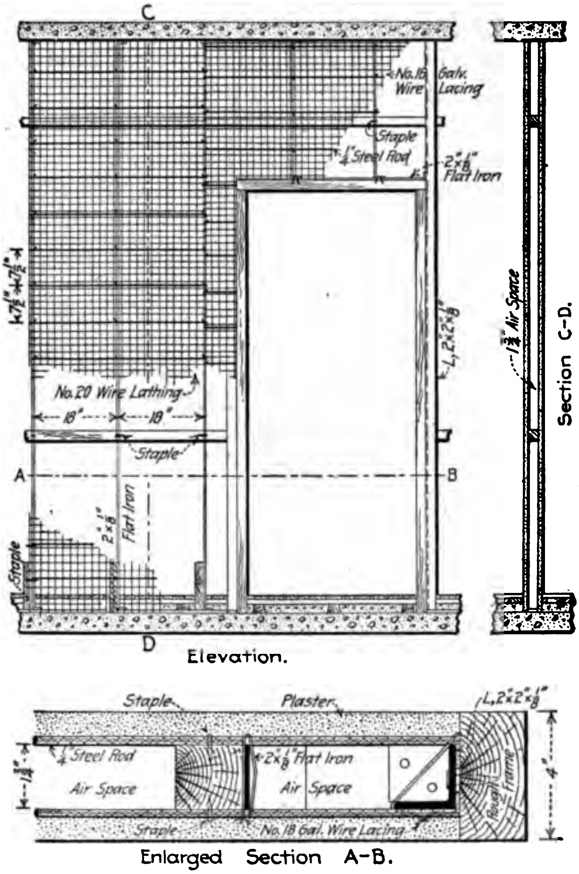


Fig. 266.—Hollow Wall Construction, Roebling Type.

hollow concrete blocks, laid up in cement, have become quite popular in certain localities for wall construction. Their manufacture is being developed as a new industry, and undoubtedly this form of building construction will be extensively used, as it is low in first cost and easily and cheaply put in place.

Roofs.—Reinforced concrete roof construction is similar in many respects to reinforced floor construction. It may consist of either reinforced roof slabs supported by steel framework or may be of monolithic construction.

Reinforced Concrete Roofs Supported by Steel Framework.—These may be of two kinds—roof slabs moulded in advance and placed upon and attached to the steel framework and carefully jointed together with mortar joints, and roof slabs built in place. Any one of the numerous slab reinforcements already described may be used in the construction of roof slabs. Fig. 367 gives details of a reinforced roof slab of this type used for a warehouse of the Chittenden Power Co., West Rutland, Vt. This construction consisted of slabs 9½ × 4 ft. and 3½ ins. thick, fastened

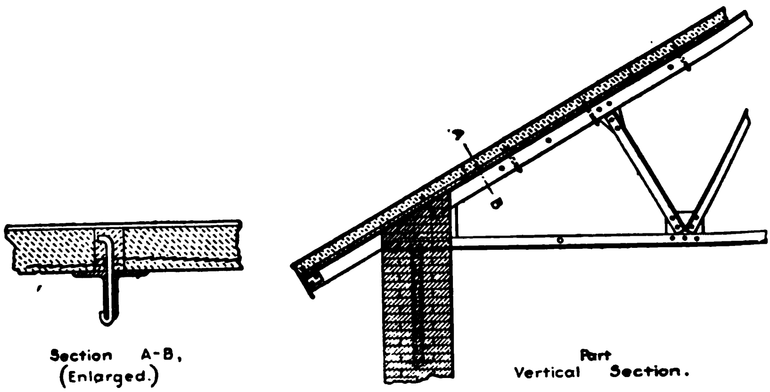


Fig. 367.—Slab and Steel Frame Roof, Chittenden Power Co.'s Warehouse.

directly to steel roof trusses spaced 9 ft. 8 ins. centers. The slabs were laid with their long sides parallel to the ridge and their ends resting on the double angle top chord of the roof trusses, so that a 2-in. space intervenes between the ends of adjacent slabs, as shown by the section A B in Fig. 367.

In constructing the slabs the reinforcement is allowed to project 2 ins. beyond the concrete, and when it is put in place the projecting edges of the reinforcement lap across the open space. Before filling in this open space short ½-in. rods were driven through the mesh of the expanded metal, which was used for reinforcement, and between the back of the angles forming the top chords of the roof trusses. The ends of the rods were then bent over, so as to clamp the reinforcement to the chord. After the clamps are placed the space between the ends of the slabs is filled with concrete, thereby securing a practically continuous slab.

The slabs were cast in special wooden forms, having their end pieces in two parts, to allow the reinforcement to project out of the forms. A ½-in. layer of rich cement mortar is spread in the bottom of the moulds and the reinforcing mesh is laid on this mortar bed. The concrete is then dumped into the forms, thoroughly tamped and trowelled smooth on its top surface. A stone concrete composed of a 1 : 2 : 3½ mixture was used.

The second form of roof construction of this type consists of

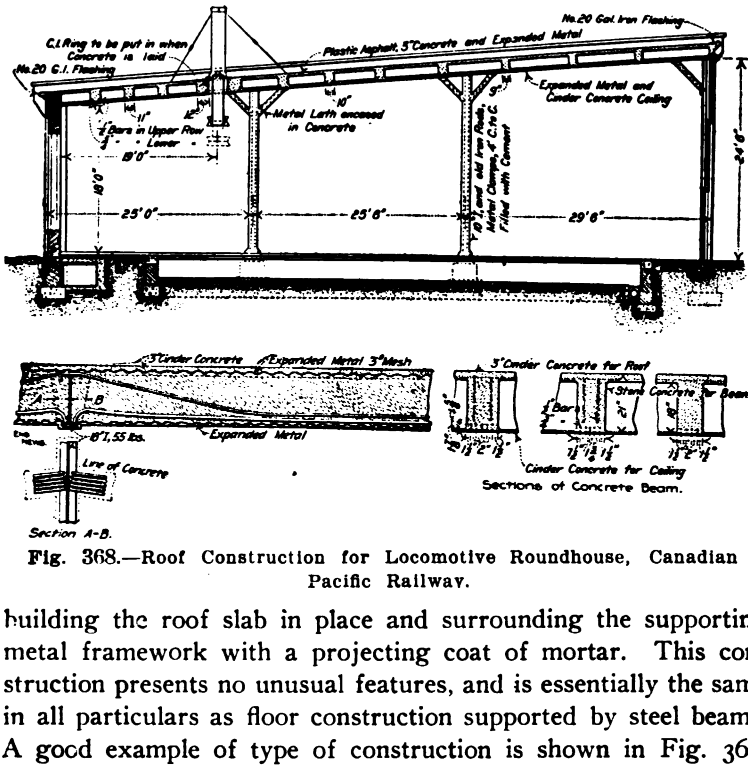


Fig. 368.—Roof Construction for Locomotive Roundhouse, Canadian Pacific Railway.

building the roof slab in place and surrounding the supporting metal framework with a projecting coat of mortar. This construction presents no unusual features, and is essentially the same in all particulars as floor construction supported by steel beams. A good example of type of construction is shown in Fig. 368. This shows details of a ribbed slab roof construction used for a round-house roof for the Canadian Pacific Ry., located at Moose Jaw, Assiniboia, Canada. The slab was of cinder concrete 3 ins. thick, reinforced with 3-in. mesh expanded metal, and the cross ribs were of 1 :3 :5 Portland cement gravel concrete reinforced with ½ and ¾-in. bars.

Monolithic Roof Construction.—In this style of construction both the roof slab and the roof framing are of reinforced concrete. This form of construction has been applied to all kinds of roofs, flat roofs, pitched roofs, arched roofs, domes, etc., some being of very elaborate construction.

The flat and pitched roofs closely resemble ribbed floor slab construction. Examples of flat roof construction are shown in Figs. 322, 333 and 323, which show respectively reinforced roofs used in the Kelly & Jones factory building, the Central Felt & Paper Co. building, and the United Shoe Machinery Co. building. The hip roof of the Medical Laboratory for the

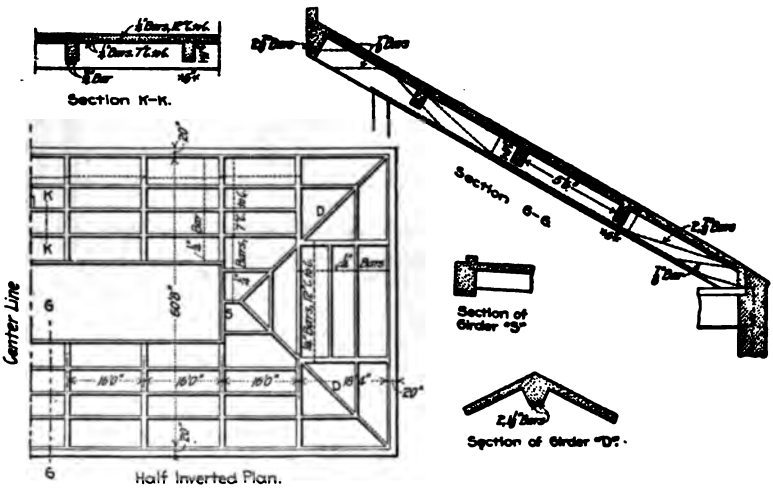


Fig. 369.—Roof Construction for Medical Laboratory Building, Brooklyn Navy Yard.

Brooklyn Navy Yard is a good example of pitched roof construction. The slope of the roof makes an angle of 30° with the horizontal. Fig. 369 shows details of this roof.

A skylight opening 16 ft. 4 in. × 79 ft. long, was provided in the peak. This required two girder rafters "S" to extend continuously over the peak between the wall plates. The intermediate rafter girders were cantilevered out 4 ft. 6 in. from the posts to support the skylight curb. A 1 : 3 : 5 cinder concrete was used throughout this roof, making a very light construction. In laying the cinder concrete a rather dry mixture was used, and it was found that no other precaution was needed to keep it in place until hardened. A slate roof covering was placed over the cinder concrete slab. The roof used for a fireproof warehouse at Los Angeles, Cal., is a good example of a reinforced concrete roof of small pitch and wide span. This roof has a center to center span of 102 ft., and is 150 ft. long. The roof slab is 4 ins. thick and is supported by reinforced concrete girders spaced 16 ft. 6 in. centers. These girders are 6 ft. 6 ins. deep at the center and have a slope of 3 ft. each way from the center.

The girders are connected by heavy curved brackets to the concrete wall piers, which are 2 x 2 ft. in sections, and reinforced by five 1⅜ in. outside and two ¾ in. inside rods. The girders are 14 in. in thickness and reinforced at the bottom with ten 1½ in. rods of medium steel, two of the rods being straight and the others bent into a hog chain form.

Stirrups of 1 in. by No. 14 metal anchor the rods securely in

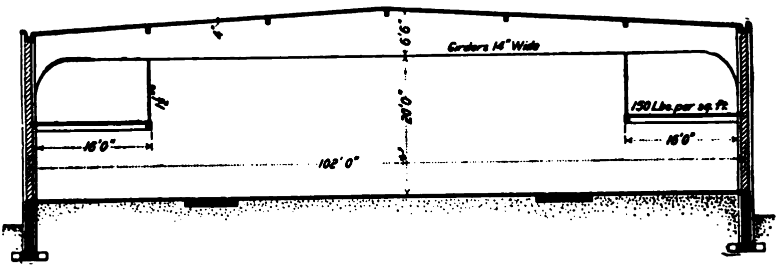


Fig. 370.—Roof for Fireproof Warehouse at Los Angeles, Cal.

the concrete. Three 1½ in, reinforcing rods 66 ft. long are used to reinforce the top of the girder.

In addition to the roof load the girders have to carry a 16 ft. gallery on each side of the building. In designing these girders provision was also made for suspended tracks for a light traveling crane. Cross beams are provided between the girders dividing the roof into square panels. These beams are 6 x 11 ins. in sections and are reinforced with four ⅞-in. rods. The reinforcement of the roof slab consists of ⅜-in. rods, 5-in. centers running in both directions. This roof is probably the widest span ever constructed in reinforced concrete. Fig. 370 shows a cross section of this building with a side elevation of the girder.

The wide span roof construction used for the furnace house of the Northwestern Ohio bottle factory at Toledo, Ohio, is an unusual type of reinforced concrete roof. This building has a total height of 52 ft. above the lowest footing. The roof girders have an out to out span of 65 ft., and are made integral with the wall columns so as to form complete transverse bents 16 ft. apart on centers.

The columns on one side of the building are 37½ ft. high from the footing to the eaves, and on the other side are 8 ft. shorter. Their upper ends are connected with the lower end of the rafters

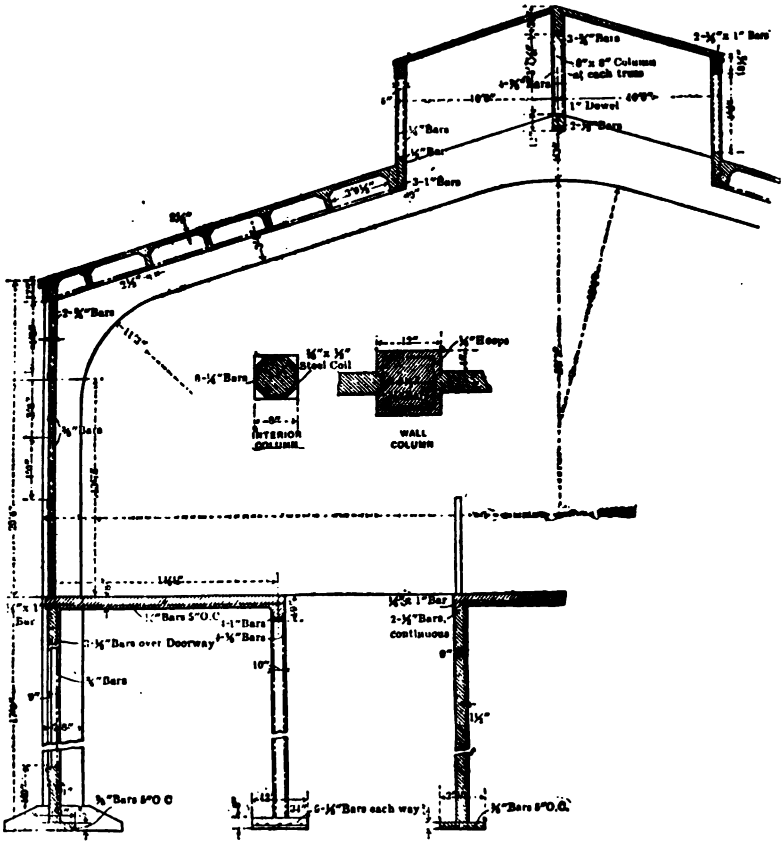


Fig. 371.—Cross-Section of Bottle Factory at Toledo, O.

by tangent curves, which, with a curve at the peak, give the straight rafters somewhat the effect of an arch. Fig. 371 shows a cross section of the building, while Fig. 372 shows the size and arrangement of the reinforcing bars of the rafter girders and columns. The rafter girders are 18 × 40-in. cross section, and are reinforced by ten 1 in. bars near the top and ten 1 in. bars near the bottom surface. The top chord bars are spliced and are made continuous to the foot of each side column. Five 1-in. bars making an angle of about 45° with the vertical extend across the foot of the rafter and top of each column, and together with transverse shear bars thoroughly reinforce the curved knee brace. The lower chord bars extend in a single length through both rafters from one knee brace to the other, being curved at the apex of the roof. They are stopped off 5 ft. from the lower

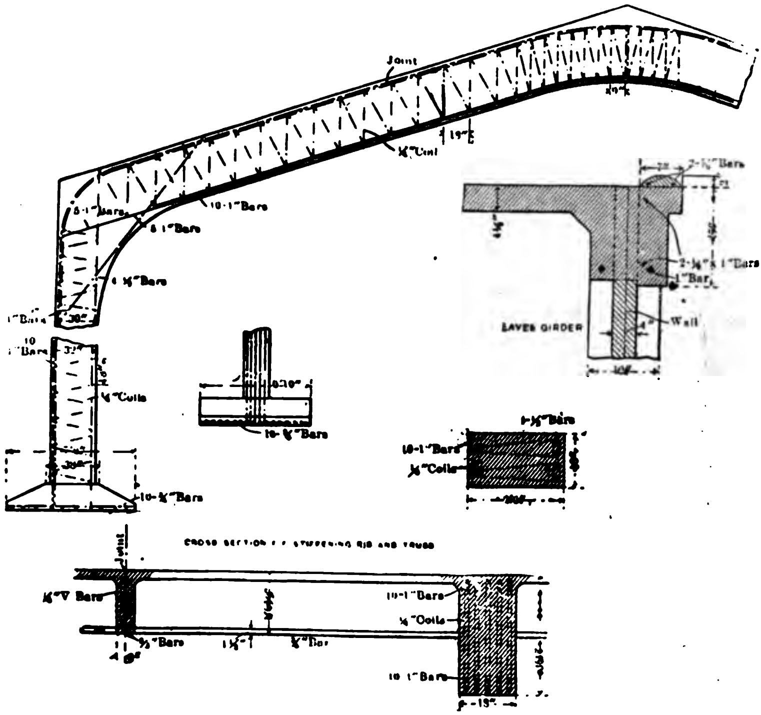


Fig. 372.—Reinforcement in Rafters and Columns, Bottle Factory, Toledo, O.

end of the rafters, where a set of five 1-in. bars in the same plane are lapped over them about 5 ft. and continue to the end of the rafters. The bars in the top chord and in the outer face of the column are made in three lengths, lapping 33 ins. and breaking joints at splices where they are inclosed by Ransome coil couplings of ¼-in. steel. Five vertical 1½-in. bars run in a single length from the footing to the top of the rafters to form the reinforcement of the inner face of the wall columns.

The column reinforcing rods are held together by a coil of ¼-in. bar with a pitch of 18 ins. The coil is made continuous by lapping the rods 12 ins. and wiring the joints.

Coils of ¼-in. rods in five sets connect the top and bottom rods of the rafters, as shown in the cross section and diagram. The pitch of the coils varies from 9 ins. at the crown to 18 ins. at the haunches. The coils are made continuous by lapping their ends 18 ins. and wiring them together. Each coil is given a full turn around the top or bottom pair of bars at every intersection, and the latter are fixed in position by spreaders between them at these points.

The rafters are connected by horizontal longitudinal purlins

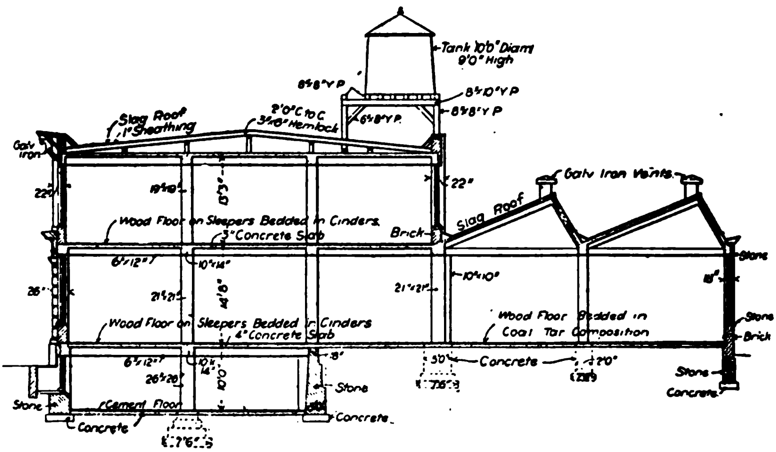


Fig. 373.—Section of Bilgram Machine Shop, Philadelphia, Pa.

4 ft. apart on center. These purlins are 2½ ins. wide and 18½ ins. deep, including the thickness of the roof slab. They are reinforced with a single ¾-in. bar 1½ in. from the lower edge, and have ¼-in. sheer bars to reinforce the webs. Transverse struts connect the purlin at the center of the panels. These are 4 ins. wide and 18½ ins. deep, and are reinforced with two ⅜-in. bottom bars. These struts are made in two equal parts with a joint at the center line. The roof slab is unreinforced. The rafters, purlins, struts and roof slabs were constructed as a monolith.

A monitor four panels long and 20 ft. wide made entirely of reinforced concrete, with large windows in the walls, is located in the center of the roof. Other features of this building are shown in the drawing and need no special description. A concrete made of 1 : 1½ : 3 mixture of Portland cement, sand and ¾-in. broken stone was used for this building. This building affords an example of a recent type of Ransome construction.

A type of roof especially adapted for machine shops and factories is the saw tooth roof used in the construction of the Bilgram Building, Philadelphia, Pa. This is shown in Fig. 373.

The roof slab is 3 ins. in thickness and supported on inclined 8 x 10 in. reinforced concrete beams. Galvanized ventilators are placed at the peak of each tooth and the skylights have galvanized iron frames embedded in the concrete. Details of construction, with the number and size of rods, are shown in Fig. 374. This building was constructed by the Reinforced Cement Construction Co., of New York.

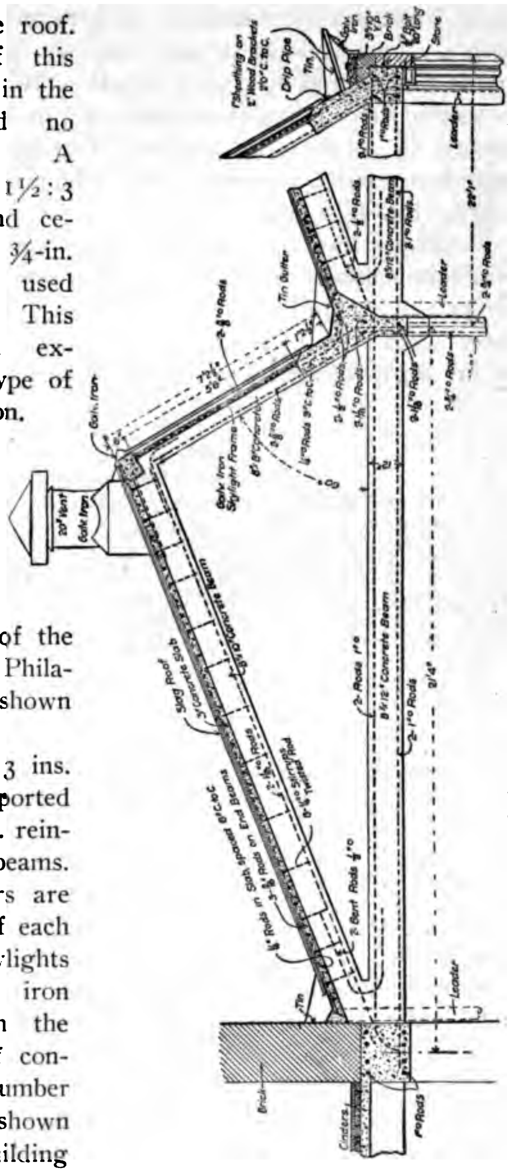


Fig. 374.—Saw Tooth Roof, Bilgram Machine Shop.

The Stamford, Conn., factory building, designed by the Reinforcement Supply Co., and erected by Tucker & Vinton, of New

York, embodies some unusual features and is of unusually bold design. This building is approximately 500 × 100 ft. in plan, and is divided into 25 × 50-ft. panels. The saw-tooth roof shown in section in Fig. 375 is supported by 50-ft. longitudinal girders carried by 15 × 15-in. colums. Transverse struts 25-ft. long span between the columns. The columns have an unsupported length of 24 ft. 8 ins.

Details of the roof framing, size, number and location of reinforcement are shown in Fig. 375; while details of main roof girder and columns are shown in Figs. 376 and 377. Figure 378 shows a view of the building in the process of construction, part of the forms being still in place. The walls are constructed of

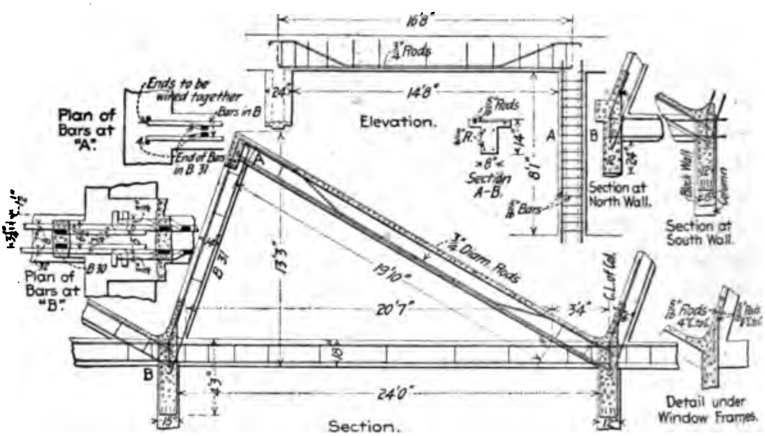


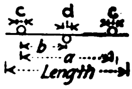
Fig. 375.—Section of Saw Tooth Roof for Factory at Stamford. Conn.

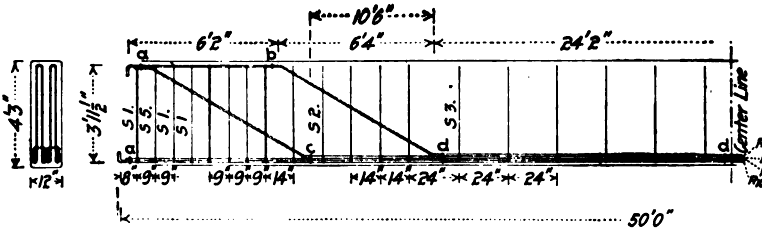
cement blocks. The Bertine system of reinforcement is used throughout.

Reinforced concrete may be used in place of timber or metal for the construction of all kinds of elaborate roofs.

In arched roofs, arch ribs or girders similar to those described under arch floors are usually employed for wide spans. For domes, either a spherical shell reinforced with rods, or some form of Monier netting, or a shell with ribs meeting at the center of the top of the dome and reinforced in the usual manner may be used. It is customary in dome construction to support the dome with a circular concrete girder reinforced circumferentially.

In Europe, Hennebique, Cottancin, Bonna, and others have





Mark.	Dia. ins.	a. ft. ins.	b. ft. ins.	c. ins.	d. ins.	Dia. ins.	Length. ft. ins.	No. per beam.	Total.
				RODS.					
R 7	....	......	......	....	....	1	51 7	4	....
R 8	....	......	......	....	....	1	51 0	4	....
R 9	....	......	......	....	....	1	49 7	4	....
R 10	....	......	......	....	....	1	50 5	4	....
				STIRRUPS.					
S 1	1 3/8	7 11 1/4	3 10 5/16	1	1 3/8	5/16	8 11	36	432
S 2	1 3/8	7 11 1/4	3 10 5/16	1	1 3/8	5/16	9 3	16	192
S 3	1 3/8	7 11 1/4	3 10 5/16	1	1 3/8	5/16	9 5	24	288
S 4	....	3 11 1/8	......	1	....	5/16	5 0 1/8	16	192
S 5	....	3 11 1/2	......	1	....	5/16	4 11 1/2	8	96
				RINGS.					
a.	....	3	......	1	....	5/16	1 4	8	96
b.	....	3	....	1	....	5/16	1 2	4	48
c.	....	3	......	1	....	5/16	1 7 1/2	4	48
d.	....	3	......	1	....	5/16	1 11	10	120

Fig. 376.—Details of Main Roof Girder, Stamford, Conn., Factory.

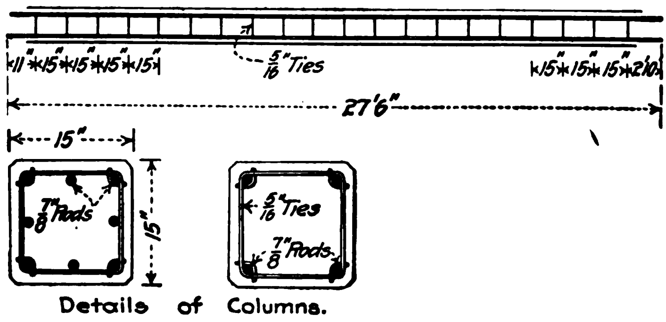


Fig. 377.—Details of Columns, Stamford, Conn., Factory.

executed many bold and elaborate designs for arched, domed and fancy roofs. One of the largest arch roofs ever built was constructed at Basel, Switzerland, for a railway train shed. This building is upward of 650 ft. in length with a width and arch span of 65½ ft. Hennebique constructed an arched roof with a span of 83 ft. on a factory at Rheims, France, in which reinforced concrete and girders replace the usual steel ones. About half of the width of this roof is taken up by skylights between the arched roof girders. In America among the notable reinforced concrete roofs are the dome of the University of Ottawa,

Fig. 378.—View of Factory at Stamford, Conn., Under Construction.

Canada; the dome for the Court House at Mineola, N. Y., and the dome of the chapel of the U. S. Naval Academy at Annapolis, Md.

The dome for the U. S. Naval Academy chapel is about 70 ft. in diameter, and consists of a shell of reinforced concrete covered with terra cotta.

Figure 379 shows a plan and section of this building with linear dimensions. The dome springs from a reinforced concrete ring which transfers its weight to 24 supporting columns resting upon another reinforced ring, which in turn is supported by lateral arches carried by 8 main piers in the main wall. The

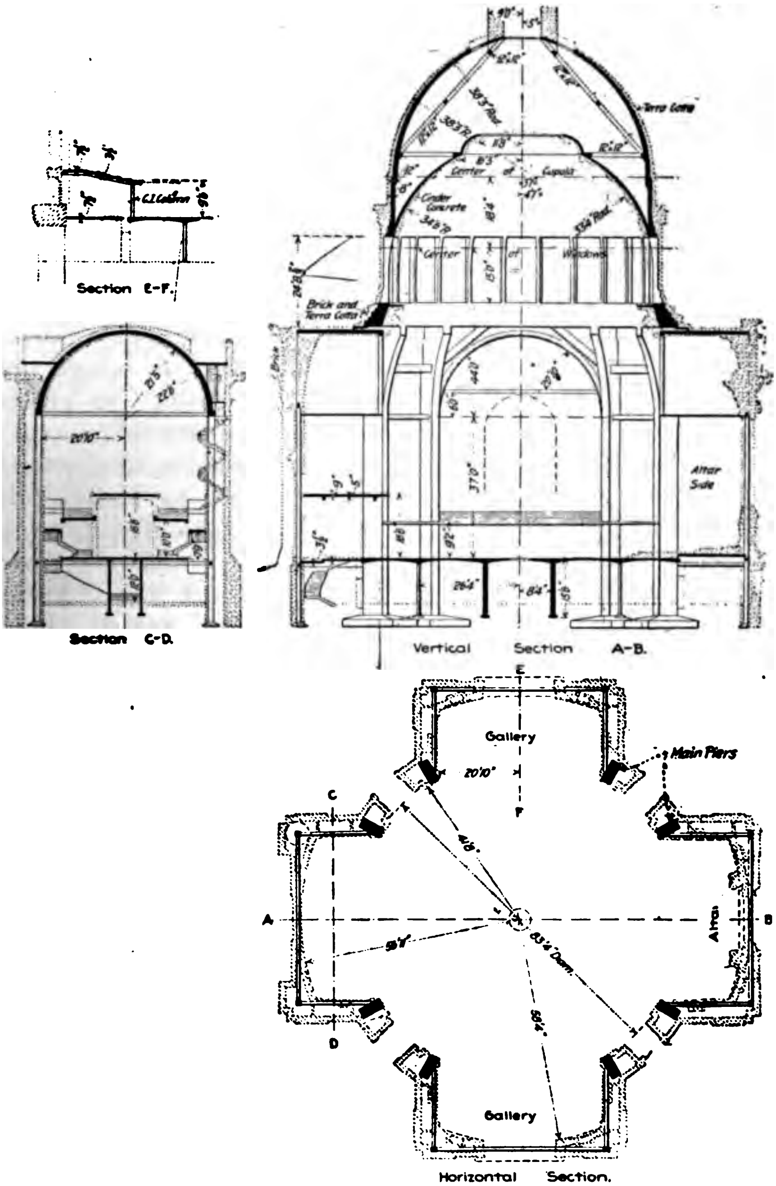


Fig. 379.—Plan and Sections of Naval Academy Chapel, Annapolis, Md.

corbeled top of the main piers with sections of circular girder are shown in Figs. 380 and 381, while half plan and half elevation with sections of same are shown in Fig. 382.

The dome is surmounted by a terra cotta lantern weighing 120 tons. This is supported independently by the pyramidal framing of reinforced concrete, shown in the cross sections, which transfers its weight directly to the walls. No permanent falsework was used except a light tower outside the building, which was used for hoisting the material. Details of framing of inner and outer shells are shown in Figs. 383 and 384. The details of falsework and moulds for outer dome shell are shown in Fig. 385.

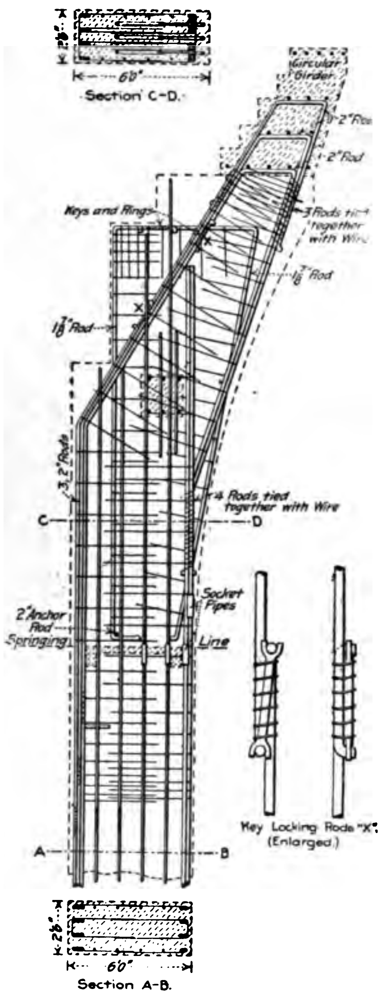


Fig. 380.—Piers and Circular Girder Naval Academy Chapel.

The forms were kept on the concrete only long enough for the latter to harden, and were then raised about two feet and clamped in position for a new batch of material. The moulds for the outside face were first put in place and then those for the inner face hung from them. The reinforcement and concrete were placed between the mould sides and allowed to harden for a few days. The moulds were then pushed forward and upward to conform with the desired curvature of the dome until, when the work had reached the crown, the men were working on a practically unsupported floor of reinforced concrete.

Reinforced Concrete Roof Trusses.—If there is any one place in which reinforced concrete should not be used, it is in the con-

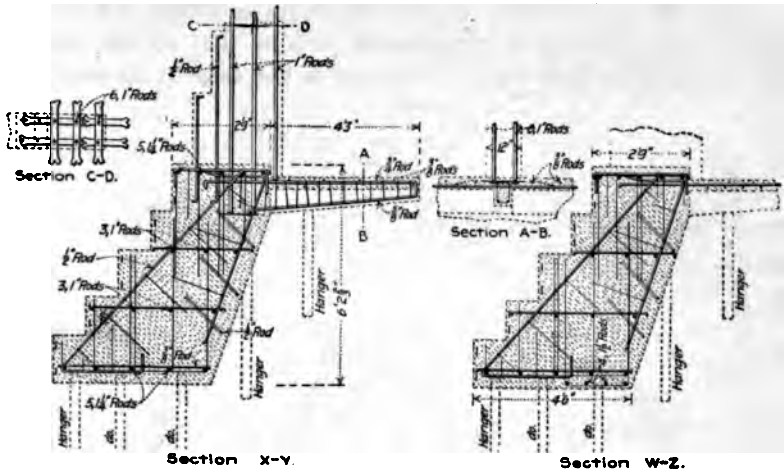


Fig. 381.—Details of Circular Girder, Naval Academy Chapel.

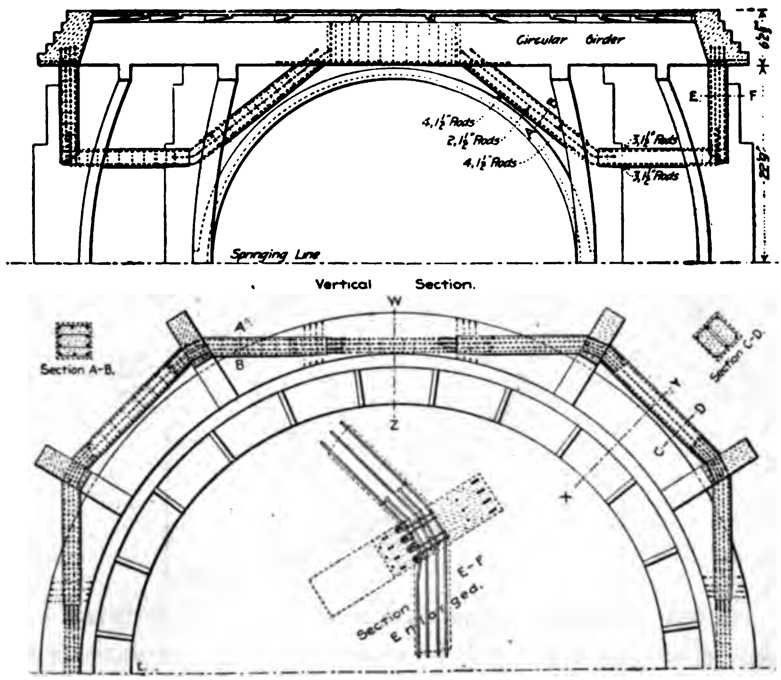


Fig. 382.—Elevation and Plan of Circular Girder, Naval Academy Chapel.

struction of roof trusses. It will be found in almost all cases that a more economical construction will be that of the usual type of skeleton steel truss. Where it is not easy to secure the

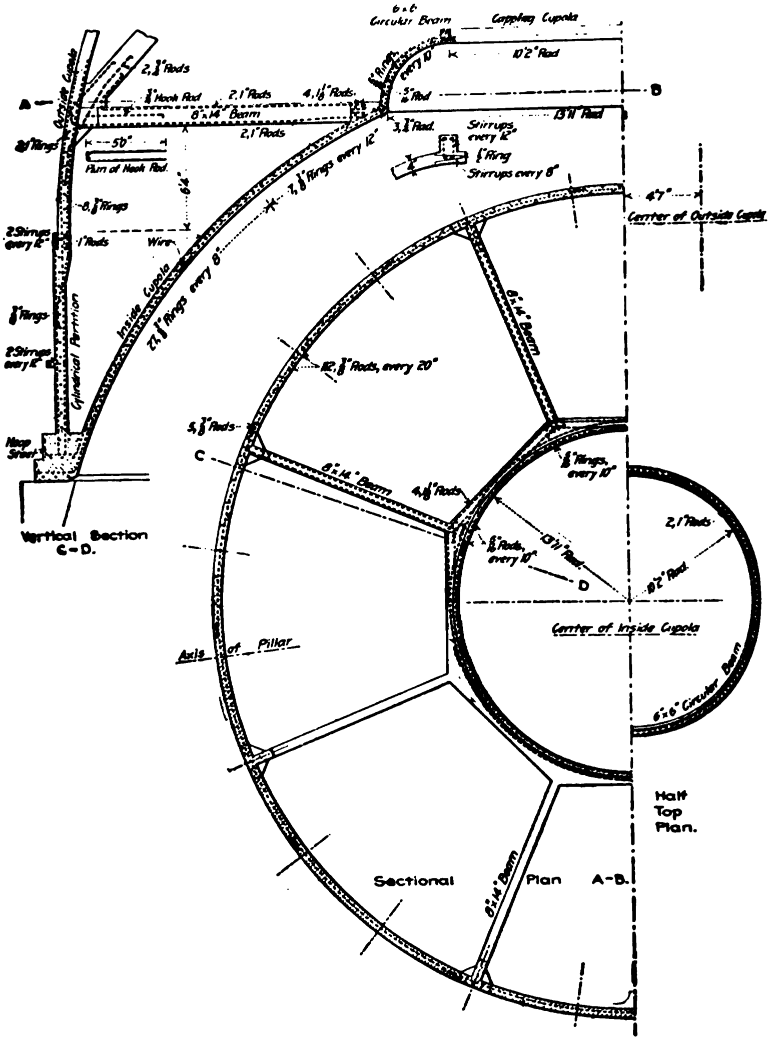


Fig. 383.—Plan and Section of Inner Dome, Naval Academy Chapel.

steel, a wooden truss may be used and will prove more economical and certainly more satisfactory when it comes to a question of analysis of the strains in the trusses and the arrangement

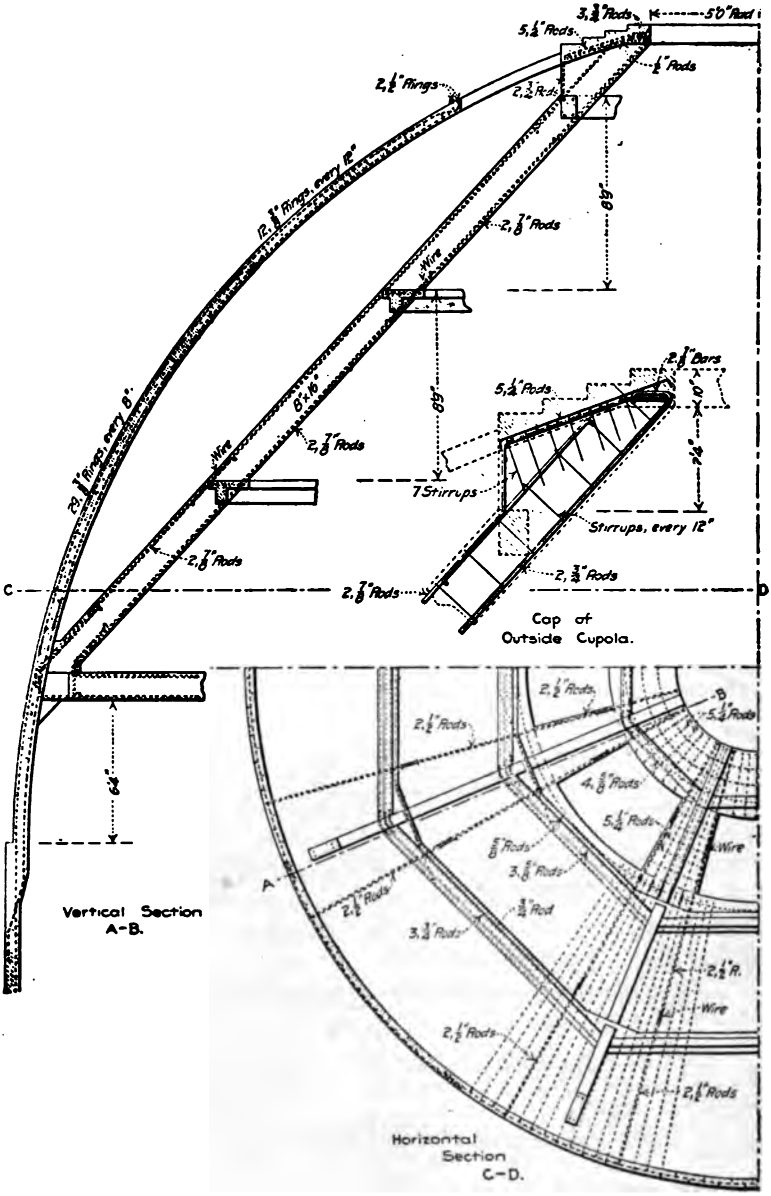


Fig. 384.—Details of Outer Dome, Naval Academy Chapel.

of the details for the joints. Reinforced concrete enthusiasts have applied this form of construction in many cases to roof

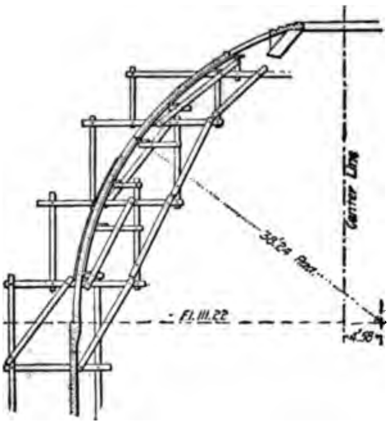


Fig. 385.—Falsework for Constructing Outer Dome, Naval Academy Chapel.

trusses. In almost all cases, however, another type of construction could with advantage have been used.

For purlins and rafters where the usual reinforced beam may

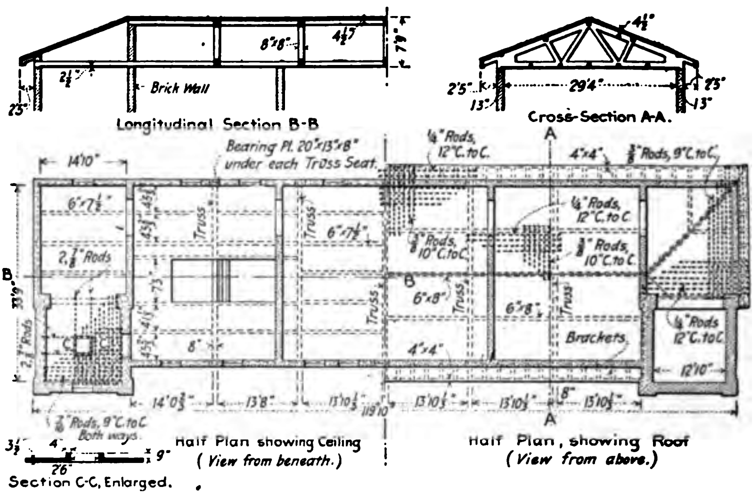


Fig. 386.—Plan of Roof Used at Atlanta, Ga., Terminal Station.

be used, and for the roof slab itself, reinforced concrete may prove both satisfactory and economical; but when it comes to complicated truss work, it is the author's belief that this type of construction should be avoided, as there is little assurance that satisfactory connections will be made to care for the stresses at the joints. However, this treatise will not be complete unless a few examples of this type of construction be shown.

In the construction of the Atlanta Terminal Station, reinforced concrete was used throughout for roof trusses, walls, floor beams, girders, columns, retaining walls, etc. Fig. 386 shows the roof construction of one of the buildings. The span between walls is 29 ft. 4 ins. The trusses were spaced about 13 ft. 10 ins. centers. Fig. 387 shows the general details, together with the reinforcing rods used for the main truss.

For the main waiting room of this building trusses of 56 ft. 4 in. span were used. The details of construction, sizes of members, reinforcement, etc., are shown in Fig. 388.

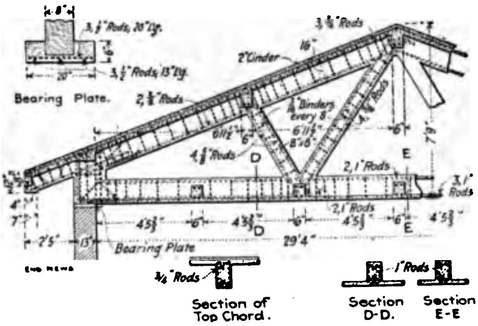


Fig. 387.—Details for Main Truss for Roof Shown in Fig. 386.

Stairways.—The strength, fireproofing qualities and ease with which it may be moulded into any desired form, makes reinforced concrete an excellent material for the construction of stairways. While reinforced concrete slabs moulded in advance and supported by a wooden or metal framework, are sometimes used in place of metal or stone for treads and risers in stairways, the more usual method is to mould the steps in the top surface of an inclined slab which may or may not have ribs of reinforced concrete for string pieces.

In the New York Rapid Transit subway stations two types of stairways are used. The first, details of which are shown in Fig. 389, is used for narrow stairways.

This stairway is supported independently of the walls of the station. Two flights, with an intermediate landing supported

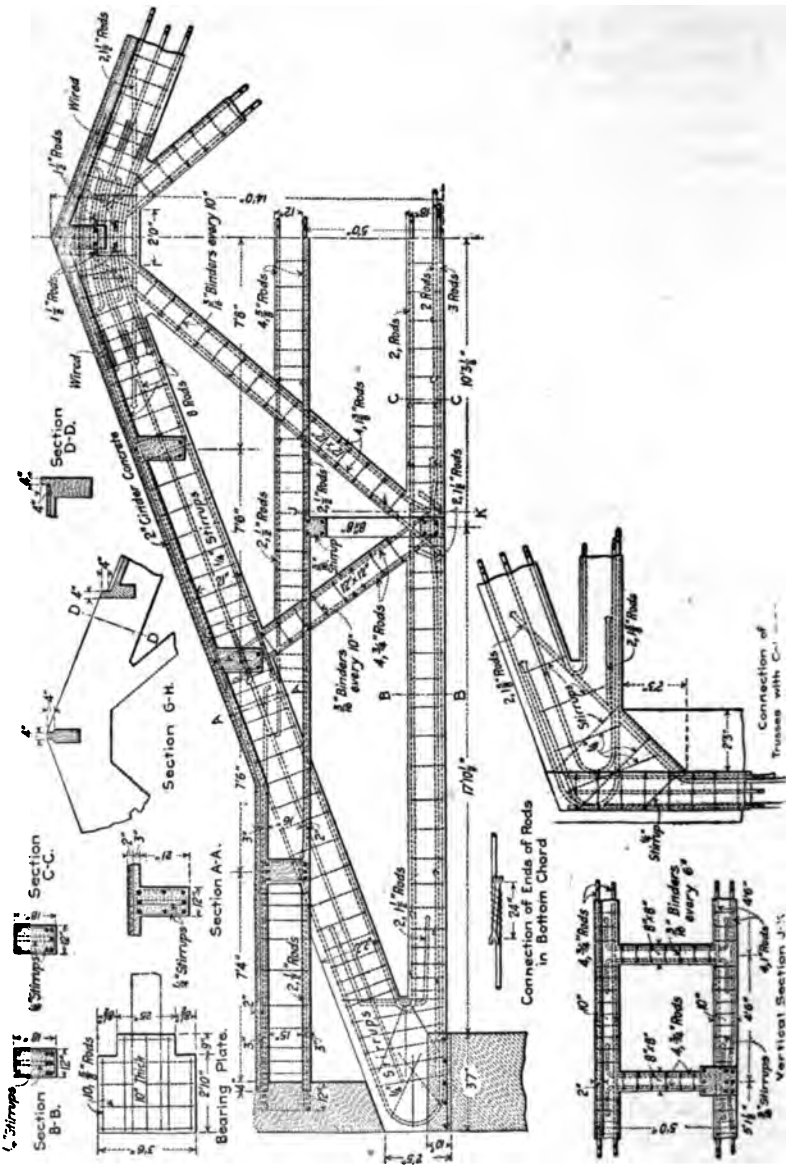


Fig. 388.—Roof Truss of 56-ft. Span Used at Atlanta Terminal Station.

by four posts, are used. The construction consists of a plate notched on the top to form the steps, and plain on the under side, carried by and built in one piece with two reinforced concrete string girders, one on each edge. These are supported on columns and on top and bottom supports. The string is reinforced with one bar near its lower edge; a ⅜-in. rod reinforces each tread, as shown in the drawing.

The second type, used for wide stairways, is shown in Fig. 390. This stairway also consists of two flights and an intermediate landing, but is without string girders. One side of this stairway

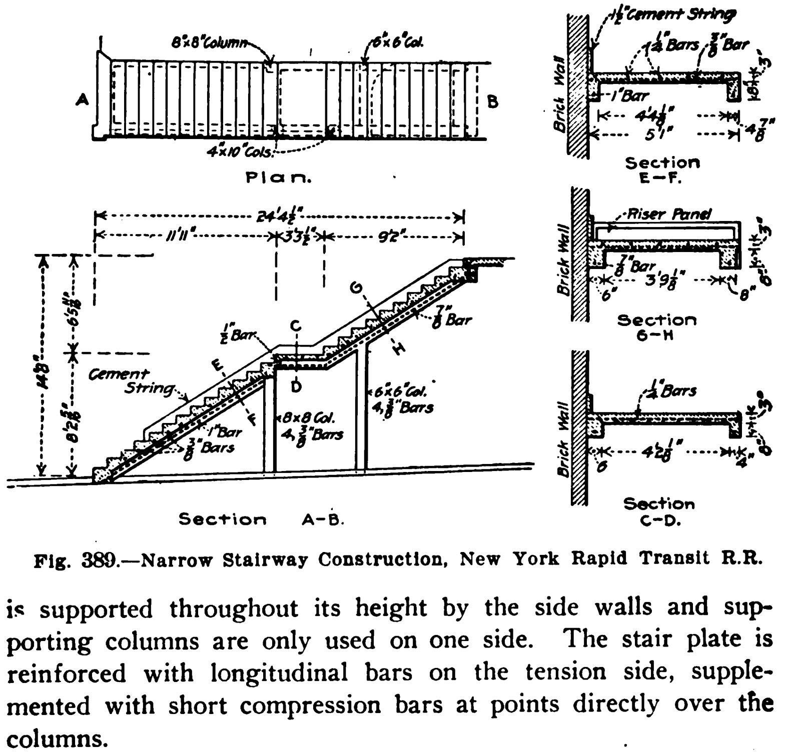


Fig. 389.—Narrow Stairway Construction, New York Rapid Transit R.R.

is supported throughout its height by the side walls and supporting columns are only used on one side. The stair plate is reinforced with longitudinal bars on the tension side, supplemented with short compression bars at points directly over the columns.

Two heavy bars run transversely through the plate over the column. The reinforcement consists of cold twisted square steel rods with an elastic limit of about 60,000 lbs. A 1:3:5 Portland cement concrete of ¾-in. broken stone or gravel was used. The panel faces and tread surfaces are finished with 1 in. of 1:2

cement and sand mortar laid with the concrete. Safety metal treads are fastened by lugs embedded in the tread surface.

The stairways used in the Medical Laboratory Building in the Brooklyn Navy Yard consist of slabs having plain bottoms and tops notched to form the treads and risers. They are all in straight runs with landings intermediate between floors. The reinforcement consisted of ¼-in. rods spaced 6 ins. apart, reaching from floor to landing, and ¼-in. transverse rods 12-in. centers between partitions.

The stairways used in both the Pacific Coast Borax Co. build-

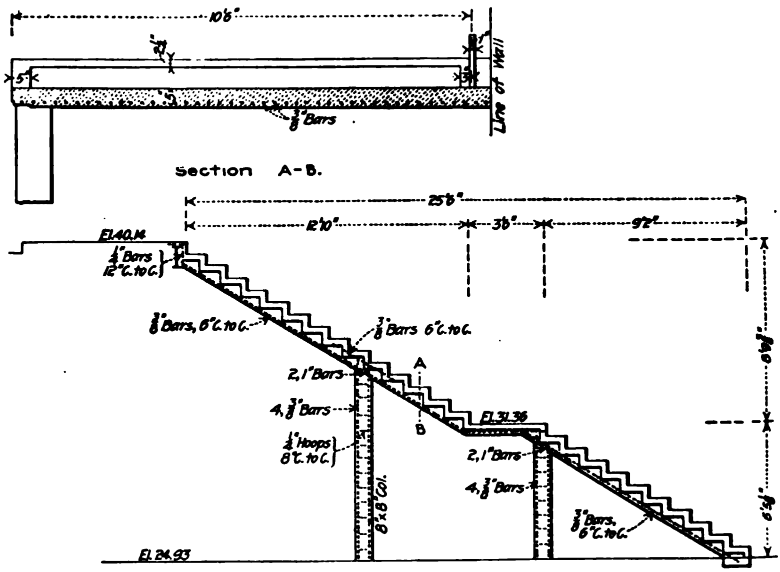
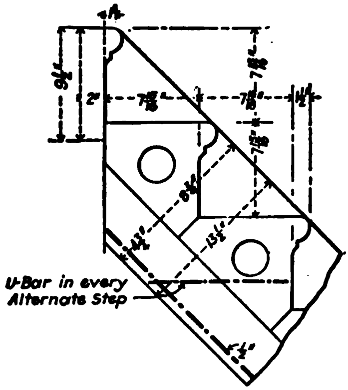


Fig. 390.—Wide Stairway Construction, New York Rapid Transit R.R.

ing and the Kelly & Jones building, mentioned on pages 499 and 500, were made with triangular horizontal steps moulded in the shops and put in position after sufficiently hardened. They were supported at each end on inclined string pieces of reinforced concrete. The details of a portion of one section are shown in Fig. 391. In the Kelly & Jones building the string pieces were inclined girders about 3 ft. deep and five ft. apart over all, and have their lower flanges flush with the under sides of the stairs and their upper edges moulded to serve as hand rails. The inner face of the 2-in. solid web is plain and the outer face is paneled. They are reinforced by a single 1-in. bar in the lower flange and a ½-in. bar in the upper flange. The lower flange is also reinforced by a 4 × 5 in. rectangular coil of ¼-in. twisted



Detail of Concrete Steps in Stairway.

Fig. 391.—Stairway with Separately Molded Steps, Pacific Borax Co.'s Factory.

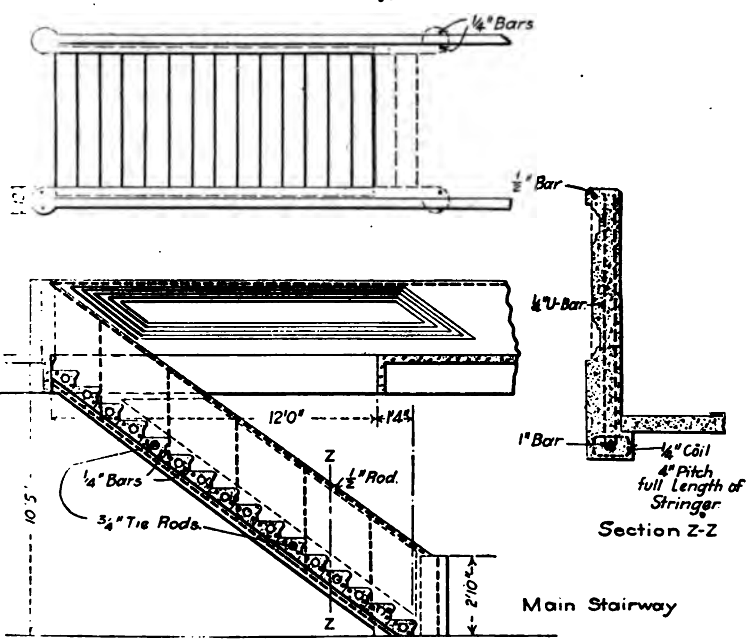


Fig. 392.—Stairway for Kelly & Jones Factory.

steel with a 4-in. pitch enclosing the tension rod from end to end. Stirrups are provided about the lower bars to take care of the shear in the stringer. The lower flange of the stringer projects inward to form a seat for the steps (Fig. 392).

A stairway of the Hennebique construction is shown in Fig. 393. The reinforcement consists of the usual straight and bent rods used in floor slabs, and stirrups at each step to tie the rein-

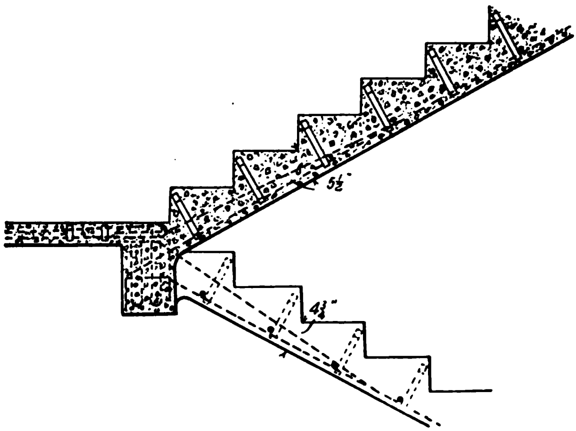


Fig. 393.—Stairway for New York City Residence.

forcement to it. This particular form of construction was used in the residence of Mr. W. C. Sheldon, New York. When overhanging stairs are desired they are cantilevered out as shown in section (Fig. 394). A large reinforcing rod is placed near the outer edge of the slab and firmly anchored at its ends. This

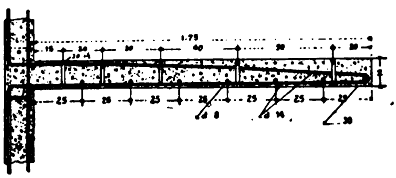



Fig. 394.—Overhanging Stairway Construction.

rod is passed through the loop at the end of the double cantilever reinforcement. Small longitudinal rods near the bottom of the slab rest upon the bottom portion of the cantilever reinforcement. Stirrups are placed at increasing intervals apart from the support toward the end.

Shaft Hangers.—It is essential to provide some means of attaching machinery and shafting to the ceilings of factory buildings. Wood and iron construction present no special difficulties, but special arrangements are necessary when reinforced concrete is used. It is desirable that the system adopted be as flexible

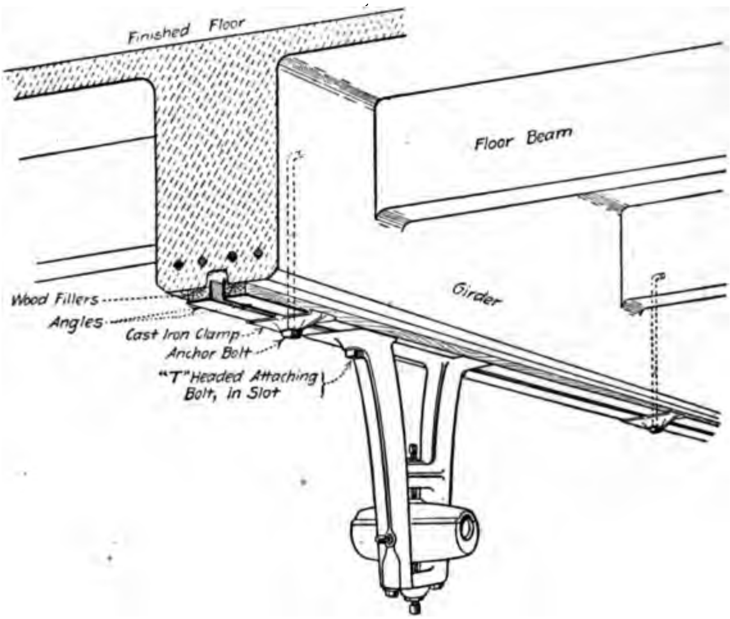


Fig. 395.—View of Angle Slot Hanger Construction for Shafting.

as possible in order to secure great freedom in the location of individual machines. This is especially desirable, as methods of manufacture as well as modern machinery are constantly being improved, and so great are the changes that new machinery or

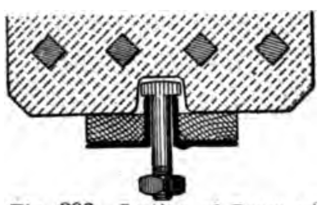


Fig. 396.—Section of Beam with Angle Slot Hanger.

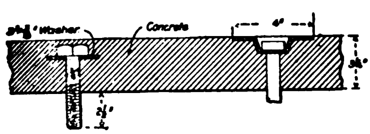


Fig. 397.—Sections of Floor Panel with Bolt and Washer Hanger.

an entire change in the arrangement of the old is often necessary. The method of attachment used in the United Shoe Machinery Co.'s building described on page 502 is the most flexible known to the author, and is the invention of Mr. H. P. Jones, of Yonkers, N. Y.

To provide fastenings for shafting, machinery, etc., anchor bolts 3 ft. on center were built into all transverse floor and roof girders. A transverse line of bolts alternately spaced 1 ft. and 6 ft. centers was built in the middle of each floor and roof panel. The bolts built into the girders had their upper ends bent at right angles and have nuts on their lower ends engaging cast-iron saddles that clamp against pairs of $2\frac{1}{2} \times 2 \times \frac{3}{16}$ in. angles with wood fillers. Figs. 395 and 396 show this

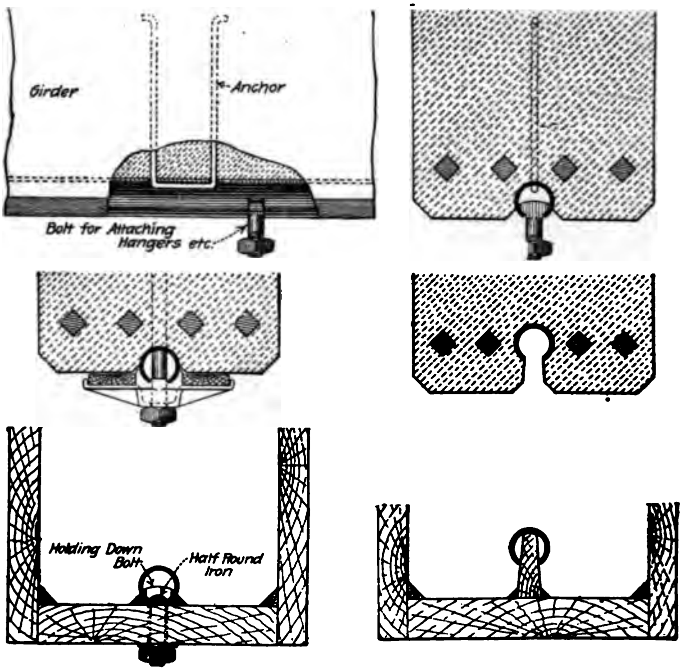


Fig. 398.—Sections of Pipe Slot Hangers and Forms for Constructing.

method of supporting the angles and attachment bolts which move freely along the slot formed by the angles.

In the floor panels the bolts have simple heads or nuts upon the upper ends, which engage washer plates in the upper surface of the slab. When the slab is thick a flat washer is buried in the slab, as shown in the left half of Fig. 397. The "U" shaped washer shown in the right half of Fig. 397 is used for thin slabs.

Figure 398 shows details of 3 kinds of pipe form slots, two of which have the pipe permanently anchored to the concrete, and the other in which the pipe may be detached by removing the nuts of the anchor bolts. The method of holding the pipe in place during construction is shown in Fig. 398. This hanger slot is also the invention of Mr. Jones. The above details will suggest other types, as wrought iron gas pipe anchored in concrete, special sockets, etc., all of which can be easily put in place and will undoubtedly prove satisfactory.

A light attachment for the support of heating or sprinkler pipes, wiring boards and light machinery is shown in Fig. 399. This hanger consists of a malleable iron casting embedded in the beam and having a T-slot in the bottom to receive the head or nut of the bolt holding the attached hanger. When a rigid connection is desired the nut is inserted in the slot. The bolt is readily put in or removed, and is adjustable for a distance of 1½

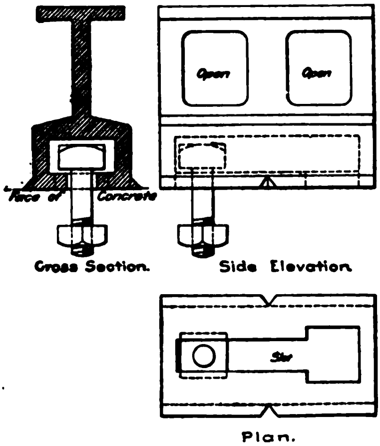


Fig. 399.—Hanger Construction for Light Loads.

or 2 inches, according to the size of the casting. When placing these castings, where possible, as in floor slabs, they are bolted to the centering. When used in beams a light galvanized strip of metal is used with holes in the ends to take two 12d. nails. This strip holds the insert casting firmly in place during the placing of the concrete. This insert hanger was designed and first used by the wholesale hardware firm of Farwell, Ozmun, Kirk & Co., St. Paul, Minn., who now manufacture and sell it.

Expansion Joints.—In the construction of large reinforced concrete buildings it is usually advisable to make some provision for expansion, although this is not always done. In the Kelly & Jones building, described on page 500, no provision for expansion was made, although the building is 60 × 300 ft. in plan and four stories in height. Expansion joints were provided in the Pacific Coast Borax Company's factory at Bayonne, N. J. This building was divided into panels 25 ft. square by joints running in both transverse and longitudinal directions. The joints in the walls extend from the foundation to the roof and pass vertically through the girders and beams dividing them into two equal parts. Fig. 400 shows a section of one pair of twin girders.

Fig. 400.—Expansion Joint, Pacific Borax Co.'s Factory.

Expansion joints were provided in the larger of the United Shoe Machinery Company's buildings described on page 502.

To provide for expansion and contraction and for shrinkage in the long side walls and floors, an expansion joint was placed at the middle and shrinkage joints were introduced at intervals between the middle and the ends. These joints, with the exception of the expansion joint at the middle of the building, were simply planes of weakness extending transversely through the building. A joint was located 50 ft. from each end, and a joint every 60 ft. between the end joints. The joints divide the transverse girders into two parts, forming twin girders, and

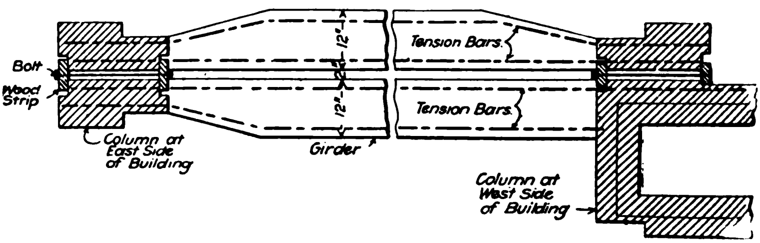


Fig. 401.—Expansion Joint in Walls, United Shoe Machinery Co.'s Factory.

were formed by building one-half of the beam, letting it set, and then building the other half against the hardened face of the concrete without any effort to bind the new work to the old. It was calculated that provision should be made for a movement of 2-in. due to expansion and contraction in the concrete between extremes of temperature. An open joint of 2-ins. wide therefore was made in a transverse plane through the walls, roof and floor at the center of each long building, dividing it into two independent halves with no connection between them other than that of the sliding cover plates which cover the joint. The opening was made through the centers of the wall columns, and at this point the transverse girders were double with a 2-in. space between them and their ends narrowed to the width of the column face and bonded to it by extending the horizontal

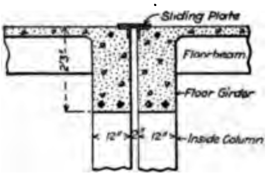


Fig. 402.—Expansion Joint in Floors, United Shoe Machinery Co.'s Factory.

reinforcement rods through the whole width of the column. This will be understood by referring to Fig. 401.

The inner and outer faces of the columns were covered by 6-in. vertical wooden strips seated in grooves 8 ins. wide and held in position by horizontal through bolts. The adjacent upper edges of the transverse girders were protected by small angles,

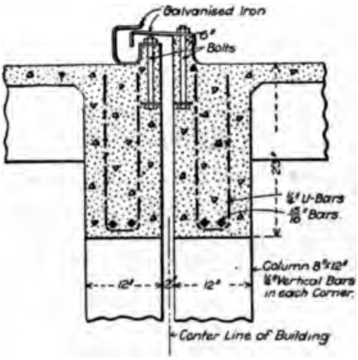


Fig. 403.—Expansion Joint in Roof, United Shoe Machinery Co.'s Factory.

one of which is counter-sunk-riveted to a beveled apron plate which rests on the other angle and slides freely on its upper surface. Fig. 402 shows detail of expansion joint in the floor.

Figure 403 shows the roof expansion joint. Transverse ridges run across the building on the faces of each joint, which rise a few inches above the general surface and have anchor-bolted to them galvanized iron bent plates which form aprons and traps to prevent a serious escape of warm air through the open cracks and to exclude wind and rain. Any melted snow which penetrates this joint is caught in the gutter formed by one piece and discharged through it to the drainage system.

Buildings Constructed of Members Moulded in Advance.—While buildings constructed of reinforced concrete members moulded in advance have been used to a considerable extent in Continental Europe, thus far they have not attained much popularity in this country. For certain classes of buildings this type of structure will be found to cost little if any more than buildings constructed of wood, and like other reinforced concrete buildings are practically fireproof. It must be remembered, however, that this type of building does not possess the rigidity which so strongly characterizes the usual reinforced concrete building constructed as a monolith, nor does it possess the rigidity of buildings constructed of wood or metal. The difficulty of securing rigid connections between girders, beams and columns is the most serious defect of this type of construction.

The greatest advantage obtained from the use of this type of building is met with when a large number of the same size of beams, columns and girders are to be used. These can be moulded in the same forms and the work carried on under cover in all kinds of weather. When the pieces have attained sufficient strength the erection can proceed with great rapidity.

A number of warehouse buildings are being put up for the Bush Terminal Co., at South Brooklyn, in which the columns, girders and roof slabs are moulded in advance. These buildings are one story in height and have brick enclosure walls. Two lines of interior columns support the roof girders and cross beams, which support reinforced cinder concrete roof slabs about 4½ ins. in thickness, also moulded in advance.

The columns are 9 ins. sq. and from 16 to 20 ft. in height, and are spaced about 17 ft. centers. The girders and beams are 6 x 22½ ins. in section and are reinforced with Bertine skeleton trusses similar to those described on page 514. In the construction of these buildings the brick enclosure walls are first built, the columns then set up, the main girders lifted in place by means of a jib crane, the cross-girders put in position and joints at column beam and girder connections grouted. The roof slabs are then put in place and covered with the usual tar and gravel roofing surface.

The Visintini system adapts reinforced concrete trusses of the Warren and Pratt type to the construction of buildings in place of the usual solid reinforced concrete beams and girders. These trusses are moulded in advance. Their lightness, on account of the saving of all unnecessary materials, make these members easy to handle. The peculiar arrangement of the members confine the stresses to the particular members in the truss which are designed to take care of them. This enables the engineer to design each member for the stress which will come upon it. Reinforcing metal is used in both top and bottom chords, but for the web members only those which take tensile stresses are reinforced. This system is the invention of Franz Visintini, of Zurich. The American rights are controlled by the Concrete Steel Engineering Co., of New York.

The first building to which this system has been applied in the United States is a four-story building for the Textile Machine Works, of Reading, Pa. This building is 200 × 50 ft. in plan. Two rows of side columns and one row of center columns, spaced about 25-ft. centers, support the transverse trusses. In a longitudinal direction the columns are spaced 12½ ft. centers. The wall spaces are filled with brick, and are pierced by large double windows in each panel.

The outer columns are 15 × 15 ins. for the first story, and 12 × 12 ins. in the upper three stories. The central row of columns are 18 × 18 ins. for the first story and 15 × 15 ins. for the remaining stories. The concrete footings vary from 6 × 6 ft. to 8 × 8 ft. in size. On top of the footing a steel plate 13 × 13 ins. by ½ in. thick was laid with a 3-in. hole in the center. Through this hole a 1¼-in. Thacher bar 5 ft. long passes, extending vertically half way into the footing and half way into the column, thereby anchoring the two securely together. The concrete of the footing was a 1 : 3 : 6 Portland cement, sand, broken stone mixture.

The inner columns are reinforced with four steel rods placed symmetrically to the center in the edges of a 10-in. square and extend in vertical lines up to the roof. They are 1¼-in. diameter rods in the two lower stories, and 1 in. in the upper. The outer columns are similarly reinforced by four rods, but these are spaced 9 ins. centers in the longitudinal axis of the building and

6½ ins. in the transverse. The sizes of the rods are the same as in the inner columns. Sleeves of 1¼-in. pipe 6 ins. long are provided to connect the abutting ends of the rods. Wire ties, spaced 1 ft. apart, are used to bind the rods together. Reinforced concrete struts tie the columns together at each floor in a longitudinal direction, and steel rods are embedded in the floor to tie them together transversely. Figure 404 shows details of one of the columns with brackets for the support of the trusses, anchor rods, etc. The transverse trusses are of the Pratt type and are 24½ ins. deep and 15 ins. wide. (See Fig. 405.) The upper and lower chords are 4¾ ins. thick and the diagonals and verticals 2½ ins.

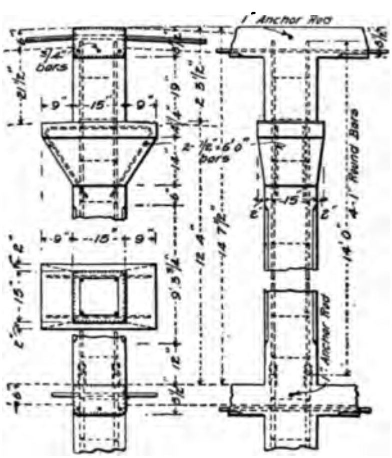


FIG. 404.—Column with Brackets to Support Separately-Molded Visintine Girders.

thick throughout. All diagonals have an inclination of 45°; this, with the distance between chord centers of 20 ins., determines the panel lengths. The chord reinforcement consists of three 1⅜-in. round rods in both upper and lower chords. The diagonal reinforcement consists of three round rods varying from ⅜ in. at the center to ⅞ in. at ends of truss. The diagonal rods are bent at the ends to form eyes through which the chord rods pass. As in the ordinary Pratt trusses, the verticals are in compression and no reinforcement is necessary.

At right angles to the transverse trusses and longitudinally with the building are placed trusses of the Warren type, which act as floor beams and floor slabs and span from truss to truss. These beams are 6 ins. thick and 12 ft. 1 in. long, and have a space of 5 ins. between the ends over the center of the transverse trusses through which the transverse tie rods pass. This space is filled with concrete. The slabs are constructed in widths of from 6¾ to 12 ins. and lie in close contact laterally and form an unbroken surface of shallow independent trusses.

The web is of the triangular or Warren type, having the diagonals inclined at an angle of 45°. The thickness of the diagonals is ¾ in., and of the chords 1 inch. The trusses are reinforced with three ¼-in. bars in the upper chord, and three ⅜-in. bars in the lower chord, and the web has members 1 in. × ⅛ in. for the tension diagonals. The metal trusses are spaced 4½-in. centers. The diagonals have holes punched near their ends through which the chord rods pass.

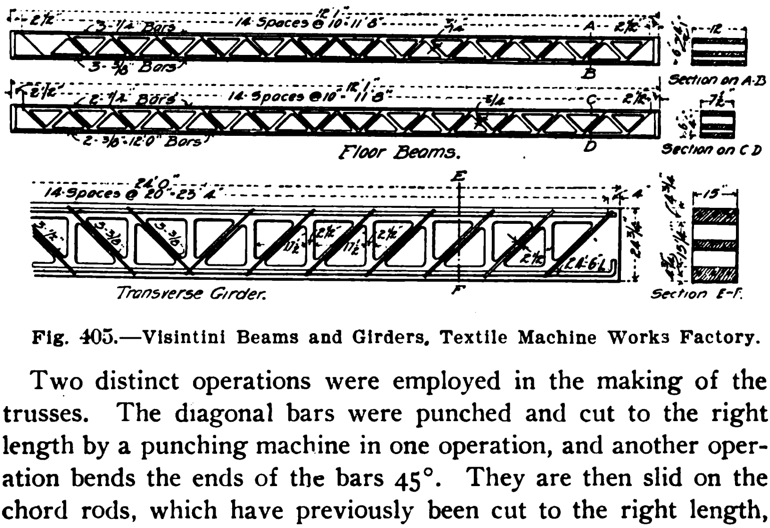


Fig. 405.—Visintini Beams and Girders, Textile Machine Works Factory.

Two distinct operations were employed in the making of the trusses. The diagonal bars were punched and cut to the right length by a punching machine in one operation, and another operation bends the ends of the bars 45°. They are then slid on the chord rods, which have previously been cut to the right length, and at their ends a vertical strap of the same material as the diagonal was put on and riveted by upsetting the end of the chord rods by a hammer. This completes a reinforcing truss for the floor beams. There were so many duplicate pieces in this building that the moulds were used many times, leading to great economy. The metal trusses, after being combined, were placed in the forms, which were composed of a bottom plank with triangular bosses the same shape as the triangular spaces in the trusses; these serve as guides for the cores around which the concrete is placed, and the cores are withdrawn after the concrete has set. The boxes are of 2-in. timber, held together by removable bolts and clamps. The concrete was mixed wet and consisted of 1 part Atlas Portland cement, 1½ parts sand and 3½ parts of broken trap rock.

The Standard Concrete Steel Co.'s System M.—This system is a distinctive system of building construction occupying an intermediate position between steel frame and reinforced concrete construction. It consists essentially of a light steel frame work of columns and beams possessing sufficient stiffness to carry the dead load of the building. Light I-beams or

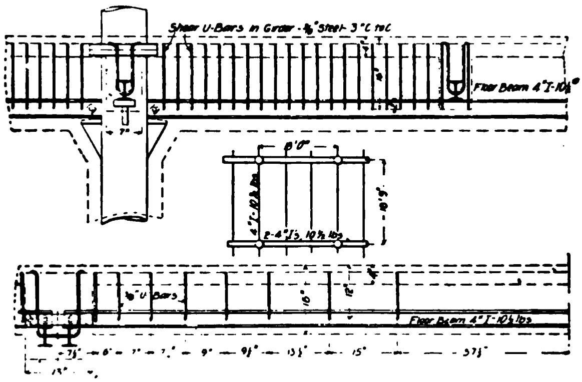


Fig. 405A—Standard Concrete Steel Co.'s System M.

channels are used in place of the usual beams and girders of steel frame construction, and are so located that they may be utilized as beam and girder reinforcement for concrete girders built about them. The beams are anchored to the reinforced concrete floor slab forming the upper flange of the concrete beams, by numerous stirrups passing through, and being firmly wedged to them and extending upwards into the slab concrete. The arrangement of beams and stirrups shown in Fig. 405A was used in the construction of the Martin abattoir, Philadelphia, Pa. The main advantage of this system is the rapidity with which it can be erected. After the skeleton work is in place concreting may be carried on simultaneously on every floor. A great saving of time results, and when rental values are high, makes the use of such a system desirable. The patents for this system are controlled by the Standard Concrete Steel Co., New York.

CHAPTER XXIV.

PRACTICAL CONSTRUCTION.

Good timbering is essential to the success of reinforced concrete work. The stability of structures built as monoliths, as well as the strength of a single piece, depends upon the strength and rigidity of the forms and falsework supporting them. The falsework in many forms of construction is very simple; it may be an integral part of the forms, it may consist of simple posts and braces as in building construction, or may involve elaborate trussing and scaffolding, as in large arch construction. It is only possible in this place to treat of it in a general way in connection with forms. Forms for reinforced concrete work must be stiff and strong enough to bear the weight of the concrete, the pressure due to ramming, and when supported, the weight of the men and materials which may come upon them, without bulging or appreciable deflection, otherwise the strength of the cement may be destroyed during the process of hardening. Forms are usually made of timber.

Timber.—It is essential that the timber selected be of good quality, so that it will not warp or twist while in place. When the forms are so designed that they can be used several times, or the timber can be used over again, perhaps a number of times, considerable outlay for a good quality of lumber will be found in the end a real saving. Under conditions where the timber can not be again used, economy, of necessity, dictates the cheapest possible material available. However, poor lumber may, if great care be not exercised, often prove to be poor economy, as its use may cause trouble. Some engineers prefer green lumber for sheathing, as it swells less than dry lumber, but when it is used care must be taken that it does not dry out and shrink after being put in place and before the concrete is deposited, thereby leaving open joints through which the water and thin mortar will drain off. When dry lumber is used, the planks should be laid up tight, but to prevent damage from swelling, it is good practice to splay the edges, as shown in Fig. 406. When treated in this manner, if the lumber swells much the thin edge will crush, without injuring the surface of the concrete.

Sheathing is sometimes laid with open joints and caulked with oakum or some other material. When thus treated the swelling of the boards does not cause trouble. Hennebique usually makes his moulds with open joints and covers them with canvas, which absorbs the excess of water, and gives a uniform surface. Sheathing should be of uniform width to obtain uniform marking of the surface when it is not subjected to special surface dressing.

Sizes of Sheathing and Posts.—In general, the following sizes may be taken as a safe guide in selecting material to be used in the construction of forms: Sheathing ⅞ in. thick should be supported every 2 ft.; 2-in. plank dressed to 1¾ in., every 4 ft.; wider spans than 4 or 5 ft. require 2¼-in. plank, and when the posts are spaced 6 or 7 ft. centers, 4-in. plank should be used. Posts and stringers of yellow or Norway pine 2 × 4 ins. to 2 × 6 ins.

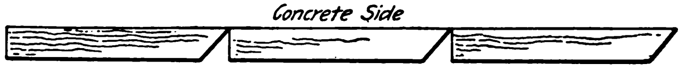


Fig. 406.—Detail of Joints for Lagging Boards.

may be used from 2 to 3 ft. centers, 3 × 8 in. may be spaced about 4 ft. 6 in. centers; 4 × 10 in., 6 to 8 ft. centers; 6 × 12 in., 8 to 10 ft. centers, etc.

Posts and beams should be sized and dressed on the side which bears against the sheathing. The sheathing should all be carefully sized to a uniform thickness and have the side next to the concrete planed. Tongued and grooved plank does not as a rule prove satisfactory for sheathing, as it is apt to swell and force out of line. Sheet-iron is sometimes used for sheathing. No. 20 gauge has been extensively used by some constructors.

Falsework.—As has been stated, the floor forms are usually supported at the ends by wall and column moulds and at intermediate points by posts resting upon the floor below. These posts rest on double wedges on the floor and bear with their upper ends against the bottom of the beam and girder moulds.

When it is possible several posts are connected by nailing diagonal and horizontal boards to the posts, thereby lining and bracing them with a stable falsework. In all cases care should be taken to secure falsework of sufficient strength and have the posts placed at such frequent intervals that no material deflection can exist between supports in the girder and beam moulds. All falsework should be so designed as to be readily removed and without bringing any shock upon the concrete which it supports. A reasonable amount of care in arranging the falsework will be well spent and may result in considerable economy.

Adhesion of Mortar to Forms.—Even smooth dressed lumber will show its grain on the concrete so close is the contact between the two. Concrete will adhere so strongly to the sheathing if no special provision be taken to prevent it, that a great deal of force is necessary to separate the forms from the concrete. Different methods are used to prevent this adhesion. The forms are sometimes lined with canvas, burlap or paper. When a superior finish is desired the moulds are at times lined with sheet-iron, zinc or plaster of Paris, but this latter method is only used in exceptional cases when the moulds will be used many times. The use of paper is not very satisfactory as it swells when damp and sticks to the concrete surface, when the cement sets, so that it is almost impossible to remove it. The most common method of preventing the adhesion is to coat the surface of the forms with some kind of heavy oil, tallow or soap. Crude oil, linseed oil, bacon fat, etc., have been successfully used. Ordinary soft soap is probably as good as anything, as it does not injure the surface or diminish the strength of the concrete. Ordinary oils will not adhere properly to sheet iron lining; fat salt pork has proved the most successful in this case. On the construction of the Frazier River Bridge, British Columbia, the lagging was laid up with matched boards coated with gloss oil, and sand was blown into the oil with a hand bellows to prevent grain marks from the wood sheathing.

Design of Forms.—Good judgment and careful planning should be exercised in designing forms. Upon this their first cost and the ease with which they can be erected and taken down will largely depend. They should be as simple as possible, should be planned to use over again several times, and when this can not be done they should be so constructed that the lumber may be used again. In building construction some preliminary study in making the layout of the building will often result in much duplication of the forms, especially in floor panel construction, with corresponding economy in the cost of construction.

While the cost of the materials for the concrete, the mixing and placing may not vary greatly, and can be closely approximated when the general conditions are known, the actual cost of the forms will be found to vary greatly. Often from 5 to 30 per cent. of the whole cost of the structure may be chargeable to the cost of the forms. In each structure the design of the forms becomes a problem for the drafting room and careful study and good judgment, when exercised in their design, will often reduce materially the cost of the whole structure, and may be the elements which determine the success of the given structure from the contractor's viewpoint, in dollars saved and profits earned for him.

Column Forms.—The construction of columns is usually carried on in one of three ways:

First—Vertical uprights are put in place and the forms built up in short sections as the concrete is put in. The uprights are held rigidly in place by frames located at intervals throughout their height. The boxing usually consists of horizontal boards cut to the proper length, forming the sides and held in place by the vertical timbers. The forms used for the Bour Building, Toledo, O., were of this kind and consisted of horizontal boards 1-in. thick and 6 ins. wide, held in place by uprights made of carefully jointed 2-in. plank. The concrete was filled in as the sides were set up and held them in place. Sometimes the side planks are nailed to the uprights and triangular strips are placed in the corners to form chamfers on the column. When this kind of form is used, a dry concrete can be put in and thoroughly tamped as the work progresses.

Second—In this form of column mould three sides of the mould are built up of heavy vertical plank, usually one story at a time. The fourth side is left open and is built up with horizontal boards nailed to the edges of side planks as the concreting is done and rammed from the open side. It is more difficult to place and tamp the concrete from one side, but the work can be brought up with surprisingly little difficulty. The vertical side planks are usually held in place by cross strips nailed to the sides at vertical intervals of about 3 ft. When it is desired to remove the forms the nails holding the strips and horizontal boards of the fourth side are drawn and the sides removed. In the Hennebique system special clamps are used to hold the sides together. This clamp, with a

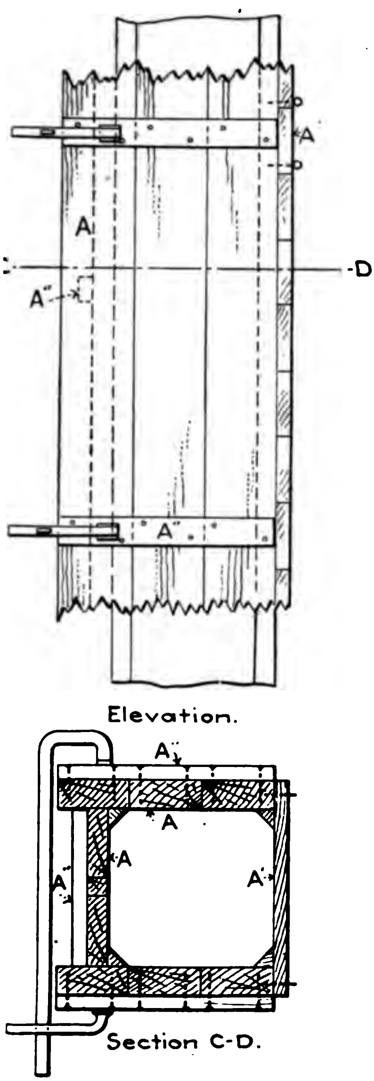


Fig. 407.—Clamp and Column Forms, Hennebique Construction.

section of the form, is shown in Fig. 407. It consists of two parts or jaws; the shorter one has a hole in its lower end slightly larger than the shank of the arm of the other jaw. The short piece is slipped upon the long arm of the other jaw and driven up snug until it jams tight and holds the sides of the form in place. When it is desired to remove the forms the nails holding the horizontal boards are drawn and the clamps knocked loose. Figure 408 shows a column form of this type, but differing slightly in construction, which was used in the building of the factory for the Central Felt and Paper Co., Long Island City, N. Y. It will be seen that this form, like the first type described above, is composed of uprights and horizontal side boards. By nailing the outer boards to the uprights, horizontal frames to hold the uprights in place are dispensed with.

Third—Bottomless boxes a full story in height built in place are much used for column forms. These may be constructed of vertical boards held in place by cross frames at about 3-ft. vertical intervals. Again they may be constructed of horizontal side pieces, held in place by uprights, which are held in place by cross frames at convenient vertical intervals.

Figure 409 shows a form used for the reinforced concrete columns used in the buildings for the United Shoe Machinery Co., Beverly, Mass. These columns were octagonal in section. The forms were made of vertical 4 × 1⅛ in. sheathing boards in single story lengths. The boards were mitered together with close joints at the angles of the columns. Each face of the mould was made of two boards jointed at the center line. At this joint the edge of one board was planed square and the other slightly beveled to make a tight joint at the inner face and an open one at the outer face of the form. This V-shape joint allows the swelling of the wood, due to the moisture of the concrete, to take place without injury to the face of the concrete. The vertical boards were nailed to the horizontal rectangular cross frames about 3½ ft. vertically. Each frame was made with a pair of transverse and a pair of longitudinal yokes and tie bolts. The 3 × 6 in. yoke pieces had triangular blocks nailed to their inner edges to re-

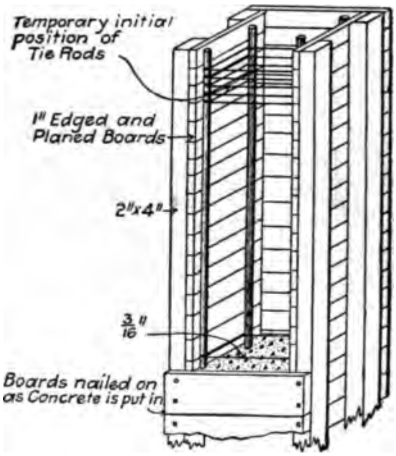


Fig. 408.—Column Form for Central Felt and Paper Co.'s Factory, Long Island City, N. Y.

ceive the corner pieces of the moulds. Tie rods, fitted with bearing washers and nuts at both ends, passed through slots in the yokes and held them together and were so arranged that as soon as the nuts were slackened the rods could be easily and quickly removed. The reinforcing rods were carefully secured in the moulds and the latter were filled with concrete; the lower portion of which was compacted by falling several feet from the top of the mould and the upper portion was thoroughly rammed and spaded.

This type of mould may be slightly modified so that it can be used for square columns by omitting the corner blocks and running the sheathing out to the corners. For columns of square or rectangular section one set of yokes may often be omitted, cross struts being substituted to hold the sheathing in place. The splayed joints should always be retained to prevent injury from swelling of the sheathing. Figure 410 shows a mould of this character used in the construction of the Ingalls Building, Cincinnati, O.

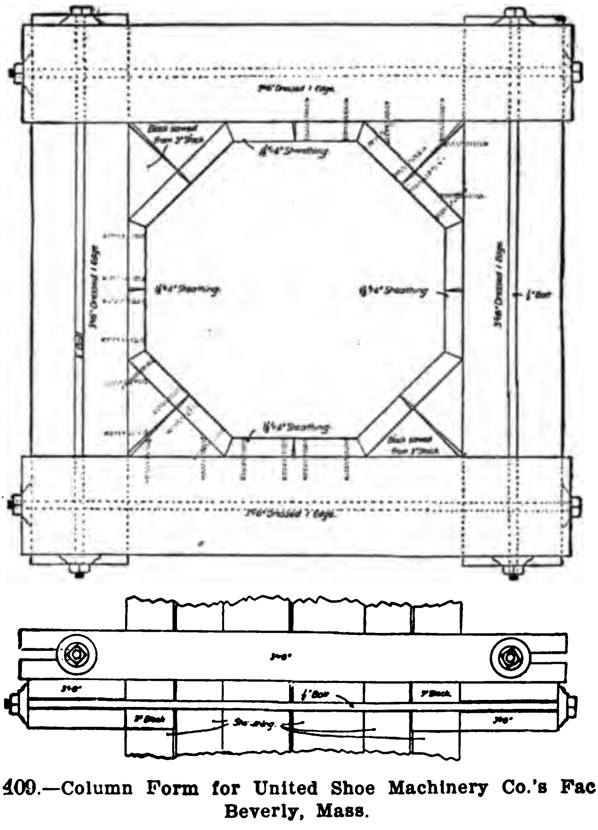


Fig. 409.—Column Form for United Shoe Machinery Co.'s Factory, Beverly, Mass.

When horizontal sheathing with uprights is used, a form similar to that shown above, in Fig. 408, should be employed.

When this third kind of mould is used, the concrete must be very wet to insure filling the moulds and all interstices about the reinforcement. It is held by some engineers that there is more or less uncertainty of securing a homogeneous concrete when this third type of column mould is used. However, very satisfactory results have been secured by others. With a reasonable amount of care it seems probable that fairly uniform results wiii be obtained and in any event it will be largely used, as it is about the only feasible method of moulding hooped columns.

In all three kinds of column moulds it is customary to attach heavy cross frames at the elevation of the cross girders and beams to support the ends of the moulds for the latter. The column mould in this event acts as a part of the falsework.

Collapsible moulds for the interior of hollow columns of more or less intricate design are sometimes used.

Figure 411 shows the core used in the construction of the Kelly & Jones factory, Greensburg, Pa. The core was collapsed by revolving the cross pieces resting in the slots about the bolts

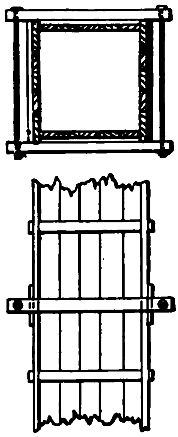


Fig. 410.—Column Form for Ingalls Building, Cincinnati, O

as an axis until it slipped free of the slots and allowed the slides tc close together.

Sometimes light centering is used for the core and left in place after the column is built.

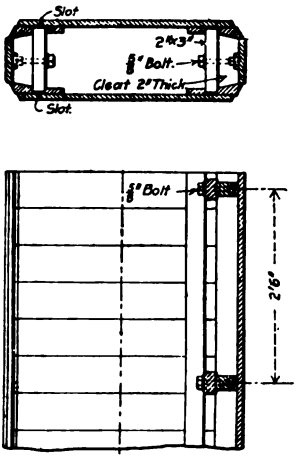
In the first two kinds of moulds the horizontal ties may be put in place as the work is brought up, but in the third style of form the skeleton work must be firmly wired in place before concreting is begun.

Fig. 412 shows T-shaped form used in the construction of the wall columns for the Parkville sub-station of the Brooklyn Rapid Transit Co.

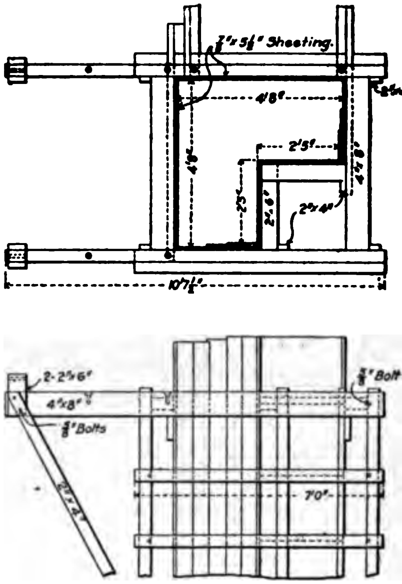
Fig. 413 shows column mould used in the construction of an

Atlantic City Hotel designed by the Trussed Concrete-Steel Co., of Detroit, Mich.

Another type of column mould is shown in Fig. 414. This

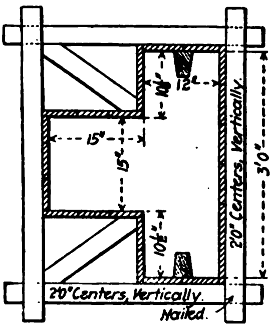


Core for Wall Column.



Mould for Corner Wall Column.

Fig. 411.—Core for Hollow Column, Kelly & Jones Factory, Greensburg, Pa.



Mould for Wall Column.

Fig. 412.—T-shaped Column Form, Parkville Sub-station, Brooklyn Rapid Transit Co.

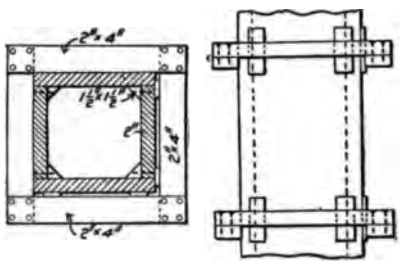


Fig. 413.—Column Form for Atlantic City Hotel.

mould was used in the construction of a water tower at Bordentown, N. J.

When fluted columns or other special surfaces are desired the interior of the mould is covered with strips of wood or plaster of Paris to secure the desired surface.

Moulds are sometimes removed in from two to six days after the concrete is put in, but it is desirable to leave them in place for two weeks, and if possible for a longer time. Hooped columns are sometimes constructed without special forms. The method of moulding columns reinforced with expanded metal, surrounded by metal lath, which acts as a mould for retaining the concrete, is fully explained on page 475 in connection with a description of the Thompson & Norris Building. Another method used in the construction of hooped columns for the Bush Terminal Co.'s building is described on page 477. In this column concrete shells are used

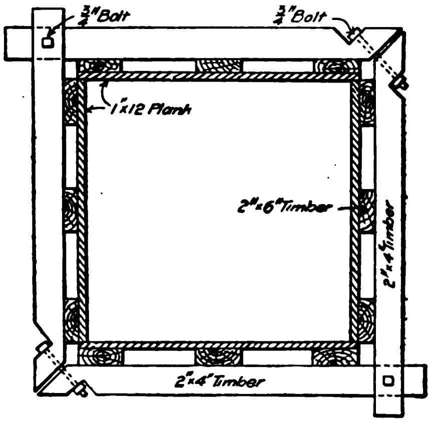


Fig. 414.—Column Form for Water Tower, Bordentown, N. J.

to act as a form for the core concrete and to hold the reinforcing metal in its correct position.

Centering for Floor Slabs Between Beams.—The construction of centers for slabs and arches used as a floor filling between beams is a comparatively simple process. The centers usually consist of flat or curved lagging carried on straight or arched joists suspended from the steel beams or girders. When the floor slab rests upon the top flanges of the beams the lagging joist may rest upon the bottom flange of the beams, as shown in Fig. 415. When the filling slab consists of either a flat floor plate or floor arch and rests upon the bottom flange of the beams the lagging is sometimes hung from the bottom flanges by hook bolts, as shown in Fig. 416. When hook bolts are used to support forms from bottom flange of beams and the bottom flange of beam is protected by a layer of concrete, the hook bolts have to be sawed off flush with the face of concrete. This may be avoided by using the form of hanger shown in Fig. 417. By greasing the bolt before it is put in place it may be unscrewed and removed.

Another form used for the floor filling between steel beams is shown in Fig. 418. This form has the lagging supported by the

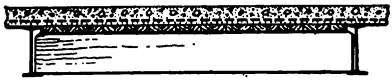


Fig. 415.—Form for Floor Slab on Top Flanges of Beam.

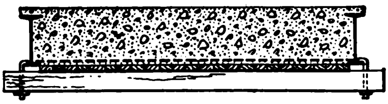


Fig. 416.—Form for Floor Slab on Bottom Flanges of Beam.

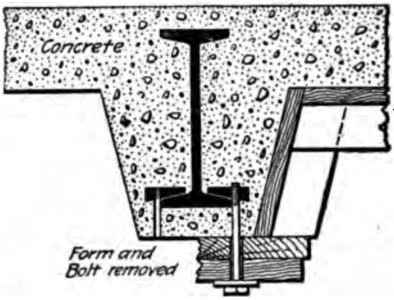


Fig. 417.—Hanger for Floor Slab Forms.

main timber A, which is 2 x 4 ins. for spans not exceeding 6 ft., and the whole is carried by the 2 x 3-in. timber B, resting upon the bottom flange of the beams. This latter piece is secured at the outer end by a cleat C nailed to A and at the inner end by cleat D, also nailed to timber A. The cleat D serves to support the battered boards and cross cleat G, carrying lagging below bottom flange of beam. The nail F is only partly driven, and when it is

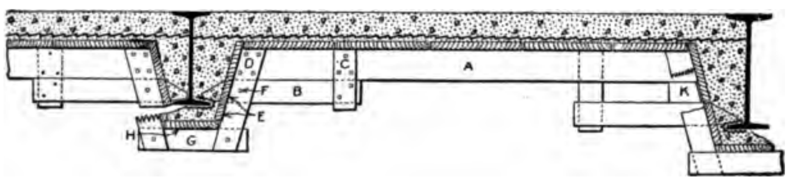


Fig. 418.—Center for Floor Slab Between Steel Beams.

desired to remove the form it is withdrawn and cleat C knocked off and B withdrawn.

When the lower flange is not fireproofed the battered lagging rests against the lower flange and part of the timbering is dispensed with. For larger beams the spacing block K shown in the right-hand part of Fig. 418 is used. For small beams B may be omitted and A allowed to rest directly upon the flange of the beam. When this is done it is necessary to saw in two the timber A to strike the forms.

Wire ties are sometimes used to support the lagging, being attached to the beams through holes punched in the top flanges, and pass around and support scantling, which in turn support the lagging.

When two Monier slabs are used it is at times customary to fabricate the lower slab in advance, then block it up to the level of the under side of the top flange, construct the top plate upon it as a center, and when the top slab has sufficiently hardened lower the bottom slab to its seat upon the bottom flange of the beams and plaster over the exposed bottom flange to protect it against fire. This leaves an air space between the two slabs. Sometimes this space is filled with a meager concrete up to the level of the second slab and the latter constructed upon it as a center.

In the construction of the Donath, Roebling and some forms of arched expanded metal floors the concrete is supported directly upon the metallic reinforcing web, and false centers are unnecessary.

After putting the centering in place the usual procedure, when the expanded metal and various forms of Monier lattice systems are used, is to place the reinforcement directly upon the lagging, then deposit a single layer of concrete upon top of it. Sometimes an effort is made to draw the netting up into the concrete by means of hooks and work the concrete under and around the metal. These are very unsatisfactory methods, as in the first case it is necessary to leave the meshing more or less exposed or plaster it over afterwards, and in the second case the final location of the meshing is very uncertain.

A better method is to place a thin layer of concrete upon the centering, lay the netting upon this, and then deposit the concrete, ramming it into place until the level of the second reinforcement is reached; put it in place and cover it with concrete until the proper thickness is secured, or when a single reinforcement is used carry the concrete to the top at once. This method is always employed when single bar reinforcements are used, care being taken to properly space the rods. In the Columbia, Müller, Wünch and Melan systems, owing to the stiffness of the reinforcement, it is not difficult to locate it in the proper position.

Upon the correct location of the reinforcement in the slab, and the placing of the concrete in such a manner that it will act as a monolith, the strength of the floor slab and the utility of the construction depends. These requirements are essential to secure the strength and stability of the construction, and too much care cannot be taken to see that they are properly carried out.

Care should also be taken when expanded metal or wire mesh reinforcements are used to cut the sheets to the proper length. If they are too long, the workmen sometimes, instead of cutting them to proper dimensions, try to force the sheet down into place, and the resulting location of the metal and its utility as a tension member becomes of doubtful value at best.

When no ceiling plate is used the lagging must be dressed smooth, or objectionable marks will be left upon the concrete. The surface of the lagging should be coated with soft soap or grease to prevent the concrete from adhering to the boards; sometimes oiled paper is used. This is very important, as every precaution must be taken to keep the centers from sticking to the concrete; otherwise difficulty will be experienced in striking the centers, and if force be used the concrete will be permanently injured. Under no consideration should the lagging be jarred loose by blows from a crowbar or sledge.

Monolithic Floor Construction.—The ordinary type of monolithic floor consists of main girders spanning between walls and columns and supporting floor ribs at right angles to them; these latter in turn support a flat floor slab.

Two methods of construction are employed: First, the forms for the girders are built up to the level of the bottom of the cross beams, and the concreting brought up to the same level; then the forms for the cross beams and girders are carried up to the bottom of the floor slab and concreted to the same level, after which the slab forms are constructed and the concreting finished. Second, the entire construction of the given floor or floor section is built up as one continuous operation and make a perfect monolith. The first method is commonly used in Europe, but in this country the more truly monolithic construction of the second method is most frequently used, and deserves its well merited popularity. While the methods in both cases are essentially similar, and the forms used vary but little in general detail, both methods will be described and illustrative examples given of forms used on important constructions.

The moulds for the girders are usually supported at their ends by the column and wall moulds. Either extensions of the frames are run out or cleats are nailed on the sides of the forms to support the ends of the boxes. Between columns the mould boxes are supported by posts resting upon double wedges for adjusting their height.

The Hennebique type of mould is constructed of a bottom piece B (Fig. 419), and supported at intermediate points by vertical posts and at the ends by the column and wall moulds. Two side pieces B′ reach up to the level of the under side of the secondary beams. These side pieces are held together by clamps, as shown in Fig. 420. The clamp shown is for use in small beam

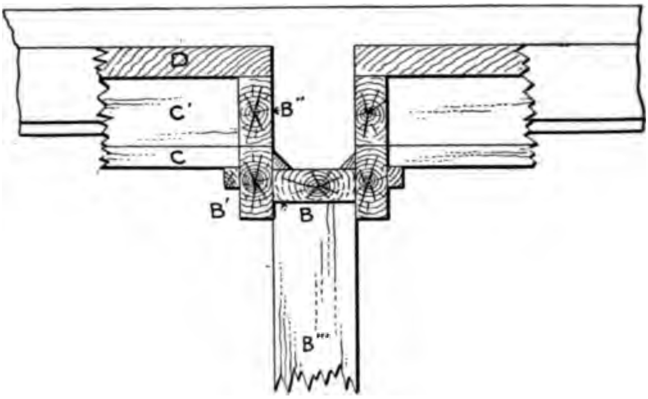


Fig. 419.—Hennebique Form for Slab and Girder Floors.

and column construction. It consists of the hook A, made from a $\frac{1}{4} \times 1\frac{1}{4}$-in. iron flat by bending one end to a curve, as shown. The dog B is of square iron, with one end slightly bent to form the jaw, and has a hole at the other end somewhat larger than the shank of the hook piece A. The dog is slipped upon the shank and tightened by hammering on the lower end until it jams. The outward pressure of the form boards upon the upper end causes it to bind and prevents it slipping back. If need be, a wedge of wood may be driven in to assist in tightening the clamp. This form of clamp is used in both column and beam construction.

It is usually desirable to construct the bottom of the girder or beam mould of one piece of timber, but when the beams are large and the span wide two or more pieces may be used, care being taken to so fasten them together that the top surface of adjacent pieces will remain flush at splice points.

Care must be taken that the top edges of the side pieces B′ are level with the lower side of the cross beams, as the sides of the boxes for these will rest upon the top edges.

Chamfers for the bottom of the beams or girders are formed by lightly nailing triangular strips in the bottom corners.

When this portion of the form has been constructed it is customary to tamp concrete in the bottom of the form to the thickness desired below the reinforcement, place the reinforcing rods in position, together with the stirrups, and hold the latter in

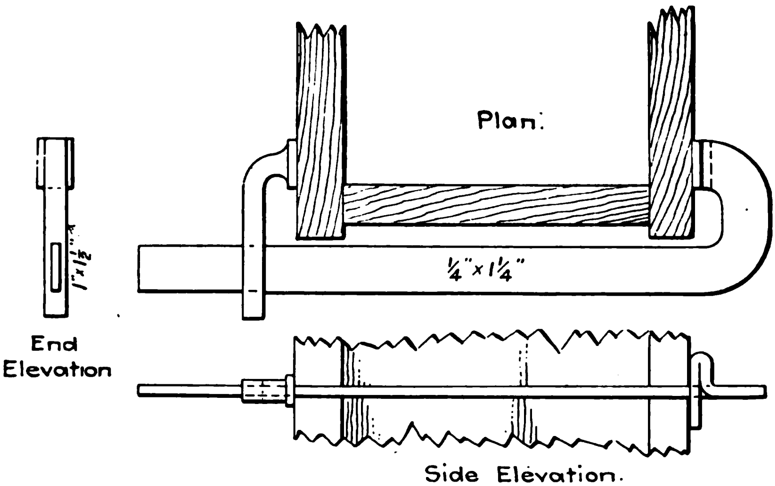


Fig. 420.—Hennebique Clamp for Floor Girder Floors.

place with small mounds of concrete; then bring the concrete up to the bottom level of the cross beams.

The bottom and sides of the moulds for the cross beams are now put in place. The bottom C is supported at the ends by cleats nailed to the sides of the main beams, and intermediate posts support it at frequent intervals.

The side pieces C′ rest on the side of the girder moulds, and are held together by clamps at the bottom and pieces of board nailed to the top edges. The level of the top of the sides C′ of the secondary beams and the additional side pieces B″ of the primary beams are made so that when the bottom boards of the floor lagging rest upon them their top surface will be at the correct elevations of the under sides of the floor slabs. The planks forming the top side pieces of the primary beams are held together by pieces nailed to the side and distance pieces nailed to their top edges.

The main and secondary beams are now moulded up to the level of the under side of the floor beams.

The floor centering is next put in place and the concreting continued as rapidly as possible. The floor lagging is supported on the top edges of the sides of the principal beam boxes and runs parallel with the secondary beams. The edges of the outside plank in each panel rest upon and are even with the inside of the cross beam sides. Cross timbers, supported at the ends by cleats nailed to the sides of the cross beam boxes, or sometimes supported by posts, are placed at proper intervals to keep the floor planks from sagging.

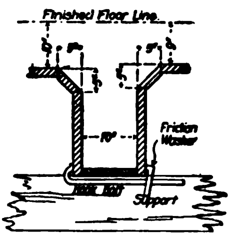


Fig. 421.—Girder Form Used in Ingalls Building, Cincinnati, O.

Care should be taken to keep the beam and girder boxes in line, and in all cases proper spacing should be rigidly maintained.

Fig. 421 shows a girder form used in the construction of the Ingalls Building. These moulds were carried on timbers between column moulds and by posts at the center span.

It is sometimes desired to remove the sides of the beam and girder moulds before the floor is constructed. In order to do this the sides of the boxes are run up to the level of the under side of the floor. When the concrete is sufficiently hard the sides of the boxes are removed and the floor lagging put in place between the beams. The lagging is held at the proper elevation by cross pieces resting upon timbers clamped at the proper height to the side of the cross beams and supported by props.

Another method is to lay transverse lagging upon longitudinal timbers supported by posts. Fig. 422 shows a form of independent mould for a floor panel. When in place the concreting is carried on as before.

Many modifications of the above described type of floor forms are used. Fig. 423 shows a modification of the Hennebique forms, in which the clips are omitted and a slightly different arrangement of the side plank and lagging used. The parts are fastened together with wood or lag screws.

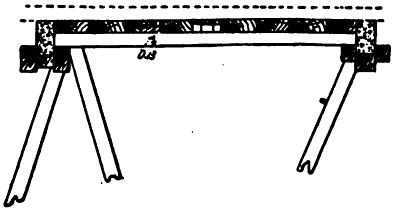


Fig. 422.—Independent Mold for Floor Panel.

The methods of construction used in the United Shoe Machinery Co.'s building are a good example of the best practice in reinforced concrete building construction. The following description of the manner of conducting this work is slightly condensed from an article in "Engineering News" by George P. Carver, Resident Engineer for this work.

After the foundation walls and footings for the columns had

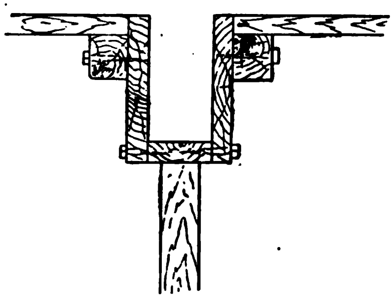


Fig. 423.—Modification of the Hennebique Forms with Clamps Omitted.

been completed the clay sub-grade of the floor was leveled and well tamped and a 4-in. layer of concrete laid and rolled for the sub-floor. When the sub-floor had set, spiral coil and rod reinforcements for the lower intermediate columns were set up and inclosed in a column form set to line and temporarily braced. The lower columns were 22 ins. and octagonal in shape. The exterior columns on one side of each building were constructed to act as hot-air flues. Before setting the forms for these columns a hollow brick lining was built up to the grade of the next floor and the brick lining was mopped with asphalt. This lining was then encased in a rectangular wooden form, allowing from 8 to 12 ins. thickness of concrete around the brick. The columns were reinforced with vertical rods and hoops placed at intervals throughout the height. The column forms used in this building are described on page 577.

When all the column forms had been set and temporarily braced the forms for the girders were set up, resting upon tall horses spaced about 3 ft. centers, the ends of the girder forms fitting with the top of the column forms. Figure 424 shows details and material for the beam and girder forms.

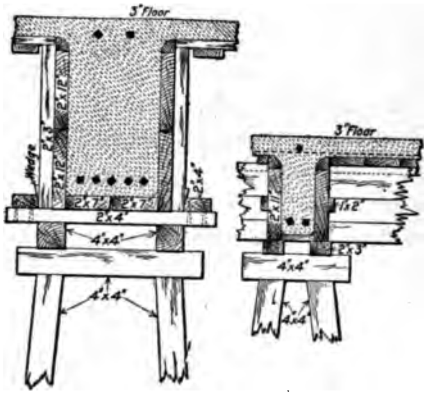


Fig. 424.—Girder Form, United Shoe Machinery Co.'s Factory, Beverly, Mass.

These girder forms consisted of a bottom board set on the horses and sides made up of short pieces, having U-shaped openings for the beam forms, which were set in a manner similar to the girder forms. The girder forms extended entirely across the building on each row of columns, which were 20 ft. apart. Midway between these girders was placed a bridging beam form entirely across the floor, and between the girder and bridging beams running across the building were set the floor beam forms, about 10 ft. long, spaced between 3 and 4 ft. apart.

The forms for the floor slab and beams consisted of bottomless boxes made with outer dimensions exactly conforming to the clear distance between beams and girders. They were spaced to secure the proper location of girders and beams and with their upper surface in the plane of the lower surface of the floor slab. Their ends and sides thus formed the vertical sides of mould for the floor beam and girders. The bottoms of these moulds were made by horizontal strips, being fitted between the adjacent boxes. The boxes are supported on their lower edges by the column moulds and on intermediate supports under the beams, as indicated in the cross section. The upper sides of the boxes are formed with 5 x 1¼-in. boards laid lengthwise, battened together on the under side and supported on cross strips 3 ft. apart. The upper edges are rounded to a 1¾-in. radius, and small triangular shaped wood fillets were placed in the lower angles of the beam and girder moulds to prevent sharp corners on the concrete.

Splayed joints between the sheathing boards were used to prevent damage from swelling. The beams and girders were so located as to secure uniform distances between them, thereby enabling the moulds to be used several times. Moulds were provided to construct a complete floor. The sides of the beam and girder forms were temporarily held apart by strips of the required length. These were taken out as the concrete was poured. When the column, girder, beam and floor forms were all set and secured to line and grade the columns were poured up to the level of the bottom of the girders and allowed to set. The columns were poured in the following manner: A mixing board was placed in close proximity to the column to be poured and a bucket of concrete from the cableway was dumped upon this board. In this connection it is necessary to state that a mixing plant was erected at one side of the work, a large Ransome mixing machine being used, and the concrete conveyed to the particular portion of the work under construction by means of a cableway. The concrete was usually very wet and of 1 : 2 : 4 mixture of small crushed stone or gravel. Previous to pouring a column the form was cleaned of all small blocks of wood, shavings and sweepings which had gotten into it. A stream of water from a ¾-in. hose was then turned into the column. The first 1 or 2 ins. of material put into the column form consisted of a mixture of 1 to 1½ grout, to make a good bond between the old and new concrete. The concrete from the mixing board was shoveled on top of this grout, and men with 14-ft. cutting tools made of wood and pointed with metal worked the concrete in between the reinforcement and the form, while a man with a heavy tamping tool rammed the concrete inside the coil. Good results were obtained, and very few voids showed in the concrete when the forms were removed. In the bottom of the girder forms and on a beveled wood strip 1½ in. high, to form a slot in the bottom of the girder, holes were bored, ¾-in. in diameter, over which were set small castings holding ¾-in. anchor bolts, which projected down through the forms the required distance, for use later as a means of supporting shaft hangers. For a detailed description of the shaft hangers used see page 563.

All surfaces coming in contact with the concrete were oiled to enable the forms to be readily removed. After the system of anchor bolts for the shaft hangers had been installed the forms were treated with a coating of crude oil and the steel placed in the bottom of the beam and girder forms. A small supporting template of concrete was made the width and shape of the beam and 1½ ins. thick and grooved on top in as many places as there were rods for the form. These blocks were used to hold the steel in position and keep it the correct distance from the sides and bottom of the forms, and were left in the forms. The rods placed in the bottom of the beam and girder forms vary in size from ¼ to 1 in., according to the depth and width of the beam. These bottom rods are of a sufficient length to have a bearing on the columns at either end, where they are lapped and fitted with a coil coupling, which consists of a coiled spring of flat band steel loosely placed around the lapped ends of the rods.

In connection with these bottom rods are placed U-shaped bar stirrups, the lowest part of which are under the tension rods. These U-bars for the most part were ¼-in. twisted steel rods, and were usually four in number at each end of a beam, to strengthen the beam for vertical shearing strains. They are placed at increasing distances from the column toward the center of the beam, as 8, 12, 16 and 20 ins., these distances depending upon the depth of the beam. The U-bars were secured to the tension rods with wire, and were held in place by a line of wire running between the tops of the same.

Great care was taken in the inspection of this work to see that the steel was clean and free from rust scales. A thin coat of rust was thought to do no harm, and probably 90 per cent. of the steel had a thin coat of rust; but if the steel was coated with rust scales before placing it in the forms it was treated in a pickling bath of sulphuric acid and water, which effectively removed the rust. In some instances the rust was removed by the use of a wire blush. When the tension members and the U-bars were in place concreting was commenced. Usually a gang of twenty-five or thirty men handled the concrete, which was delivered in buckets by the cableway. Sections 50 and 60 ft. long were poured at one time, this being the distance between the expansion joints, as explained on page 566. When the pouring was begun a 1 : 1½ mixture of grout was first used to fill in around the steel in the bottoms of the forms. This grout was well worked in and cut with tools so that every part of the steel rods would be covered with concrete. The 1 : 2 : 4 fine gravel or crushed stone mixture was then shoveled from the mixing boards into the beams and girders on top of the steel and continually cut and tamped with a tool shaped like a hoe with the blade turned down in line with the handle. In this manner the concrete was brought up to the top of the beam, when the cantilever rods were placed. These rods were laid along the beam, the centers of the rods being on the tops of the columns and with their ends abutting. After placing these cantilever rods and embedding them in the concrete, the beams and girders being brought to this height, a layer of concrete, varying in thickness from ½ to ¾ in., was spread over the floor area, and on this were laid ¼-in. rods, spaced 1 ft. centers and running across the floor. The ends of these rods lapped 9 ins. usually over a beam. On top of these rods were placed a layer of concrete of sufficient depth to bring the floor slab up to within 1 in. of the finished grade. This layer was well rolled with a metal sheathed roller about 30 ins. in diameter and weighing 250 lbs. The wearing surface was laid on this base within an hour or two after the floor was poured. The wearing surface was screeded to a level surface and then troweled until a smooth surface was obtained. In hot or rainy weather the floor surface was protected by a canvas covering spread over a temporary framework over the section being finished. After the wearing surface had set it was spread over with a layer of wet sand, the sand being kept wet continually for a period of about ten days. The moulds were removed at the end of fourteen days, cleaned, repaired and set up for the next floor. Intermediate posts were retained under the middle of the beams for several days longer

A very careful supervision was carried on by a corps of able inspectors. This resulted in securing a very satisfactory class of work.

Figure 425 shows a girder and column forms used in the con-

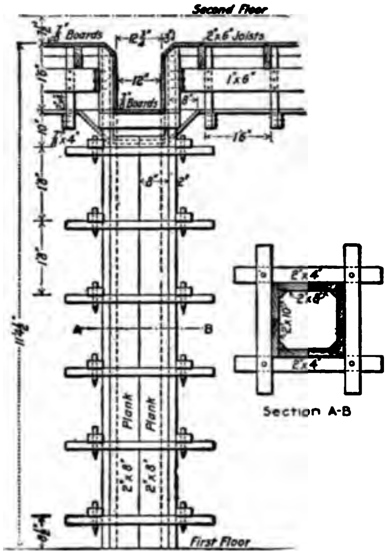


Fig. 425.—Girder and Column Form for Minneapolis Warehouse.

struction of a warehouse at Minneapolis, Minn. The materials used are fully shown in the figure. These forms were light in weight and easily erected and taken down.

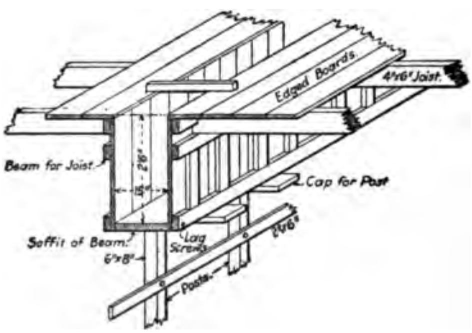


Fig. 426.—Slab and Girder Floor Mold, Central Felt and Paper Co.'s Factory.

Figure 426 shows girder and floor mould used in the construction of the Central Felt & Paper Co.'s factory, Long Island City, N. Y. Details of the construction are shown clearly in the sketch. By the withdrawal of the lag screws the bottom pieces were easily detached from the vertical sides. Both the bottom and side pieces were supported independently on transverse cap planks resting upon 6-in. × 8-in. posts spaced 6 to 8 ft. apart on the center line of the girders. The bottom piece consisted of a single plank from 1 to 3 ins. thickness; the side pieces of 4 × 7/8-in. boards nailed to 2 × 4-in. top and bottom horizontal pieces. A third 2 × 4-in. horizontal piece was placed near the top to support the joist carrying the lagging of the floor slab mould. Vertical strips under the ends of these joists transmitted their load to the lower horizontal strips and post caps. The lower edges of the side pieces were temporarily screwed to the bottom pieces, and the upper edges spaced by cross-ties, from which the reinforcing rods were hung until the concreting was begun. The first boards of the floor slab lagging were nailed to the edges of the side pieces and to the ends of the joist; the other boards were laid loose.

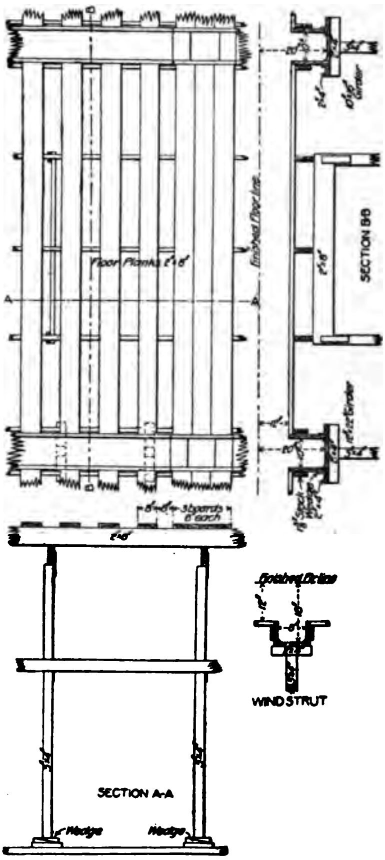


Fig. 427.—Beam Mold and Slab Center, Atlantic City Hotel.

After the floor slab concrete had set about seven days, the floor joists were turned on their side faces and the lagging lowered a few inches below the ceiling. A few days later the side pieces of the girders were unscrewed and removed, the bottom pieces being left in place some three weeks longer, until the concrete had thoroughly hardened.

In this construction the reinforcing rods were wired together and fixed firmly in place in the moulds before the concreting was begun and so held that they would not be displaced by ramming.

Figure 427 shows beam moulds and centering used for supporting the hollow tile and reinforced concrete floor construction used

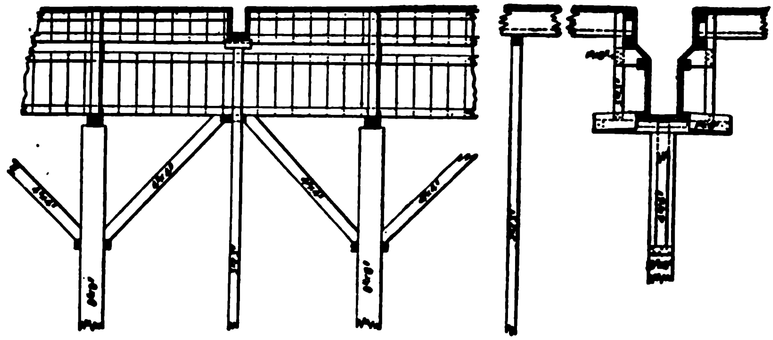


Fig. 428.—Girder Forms, Central Pennsylvania Traction Co.'s Car Barns

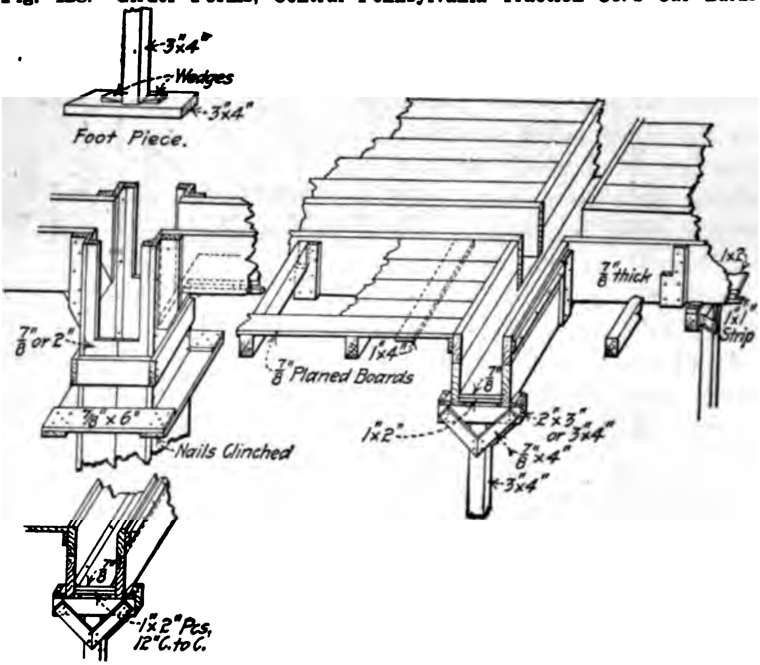


Fig. 429.—Slab and Girder Floor Forms of Unit Concrete Steel Frame Co.

in the construction of an Atlantic City hotel, designed by the Trussed Concrete-Steel Co., Detroit, Mich.

Figure 428 shows the forms and centering supporting them used in the construction of the shops and car houses of the Central Pennsylvania Traction Co., Harrisburg, Pa.

The framing of forms for columns, girders, beams and slabs shown in Fig. 429 are recommended by the Unit Concrete-Steel Frame Co., of Philadelphia, Pa., to whom the author is indebted for the drawings. The method of framing and wedging scaffolding to support forms is also shown. The posts may be loosened by knocking out the wedges upon which they rest. The side pieces of girder forms are also released by driving out the bottom wedges. The side pieces may then be removed and the bottom plates left in place.

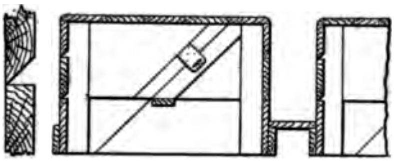


Fig. 430.—Slab and Girder Floor Forms, Pacific Borax Co.'s Factory.

Fig. 430 shows the form of moulds employed in the construction of the floors in the extension to the Pacific Borax Co.'s building, Bayonne, N. J. The mould consisted of collapsible bottomless boxes having solid sides and tops, and exactly corresponding in shape and dimensions to the cavity of the panel between each pair of girders and floor beams, which they connected.

A number of these moulds were set side by side, with their ends resting on stringers carried by column moulds and with spaces between each equal in width to the thickness of the floor beams.

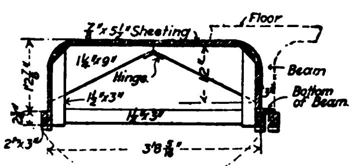


Fig. 431.—Slab and Girder Floor Mold, Kelly & Jones Factory.

The bottoms of these spaces were closed by boards, resting in cleats nailed to the side of the boxes. To collapse the mould it was cut in half diagonally. These two parts are clamped together by a slotted plate, which, when loosened, allows the parts to slide past each other and partly collapse the mould.

Another form of collapsible mould, used in the Kelly & Jones factory building, mentioned on page 500, is shown in Fig. 431. This mould is in reality a core between the beams and girders, and consists of a horizontal top and four vertical sides connected with rounded and beveled corners. The side pieces are nailed to vertical cleats and have stiffening strips on the lower edges. The moulds are made in two equal parts, with a hinged joint through the longitudinal center line of the upper surface. When put in position the upper surfaces are in the same horizontal plane and the mould is held open by transverse struts between the lower edges. The bottom of the beams are formed by horizontal boards resting upon the flange pieces of the sides of the mould boxes. These boxes are supported in the usual manner by the column mould at the ends and intermediate posts.

When it is desired to remove the moulds the cross bars are

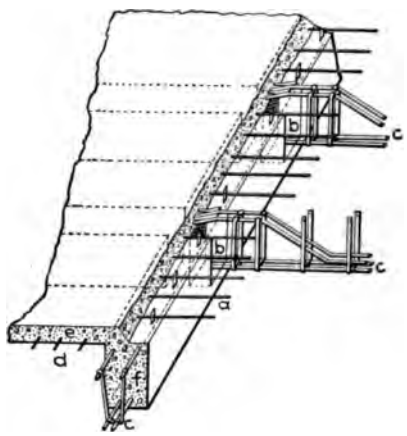


Fig. 432.—Method of Stopping Work on Concrete Slab.

knocked loose in the bottom of the mould, and a few taps cause it to close enough on its center hinges to collapse. When freeing itself from the concrete it fell and was caught on horizontal ropes stretched for the purpose.

When provision is to be made for expansion in the floor a joint is sometimes made in the floor slab on the center line of a beam or girder, and when double girders are constructed for the same purpose only a slight modification of the usual type of mould is needed. In the case of twin girders a partition cutting off half of the mould is used. The concrete is put in up to this partition, which is removed when it has sufficiently hardened; then the concreting is continued. To make a more positive clearage plane, paper or canvas may be placed at the division plane against the concrete first put in and the concrete laid up against it. This will break any possible surface bonding.

Concreting.—While no special instructions, other than those already set forth, need be given for concreting floors, it will be well to emphasize a few points, as careful and conscientious work is necessary to secure good results. The mixing should be thorough. A wet mixture is to be preferred, but there is danger in using too much water. Care should be taken in placing the concrete. Tamping should not be neglected, and when the mixture is too wet to successfully tamp it may be cut and spaded, either with an ordinary spade or one made for the purpose. A special tool, having a 6 × 8 × 1/8-in. blade attached to a 5-ft. gas-pipe handle, is used by Ransome companies in much of their work.

When the concrete is tolerably dry it is sometimes compacted

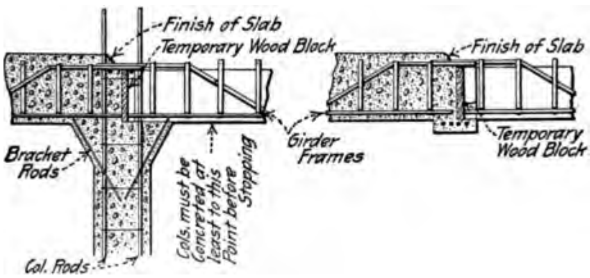


Fig. 433.—Method of Stopping Work on Beams and Girders.

by rolling. In the Kelly & Jones factory, at Greensburg, Pa., a 3 × 3-ft. 250-lb. wooden roller was first used; then a 2½ × 2½-ft. 700-lb. iron roller, for compacting the floor slab. Very satisfactory results were reported.

In stopping the work for the night the concreting of principal beams should be left off over a column as near the center line as possible. Secondary beams should be left off at the principal beams, but in all cases the principal beams should be moulded through at one time. In like manner the concreting of the floor slab should be left off at center line of the beams. The method recommended by the Unit Concrete-Steel Frame Co. for stopping off concreting when work cannot be carried on continuously is shown in Figs. 432, 433 and 434. Fig. 432 is a part plan and Figs. 433 and 434 are sections showing manner in which concrete work should be stopped. It is desirable to make the operation of concreting as nearly continuous as possible, as only by so doing will a perfect monolith be secured. In no case should the concrete be stopped before the full thickness of the floor slab is secured over the entire surface laid for the day's work.

The concrete in the floor slab should be permitted to set for at

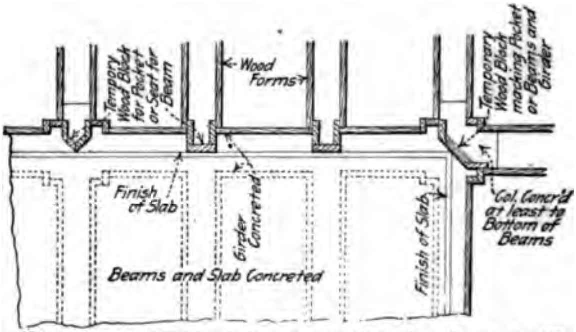


Fig. 434.—Arrangement of Forms for Stopping Concrete Work.

least a week before removing the frames; two weeks would be better. When the lagging is removed an occasional plank near the center of the span should be left in and supported by a post. The sides of beam moulds may be removed soon after the floor slab moulds are taken down, but the bottoms of the beam moulds

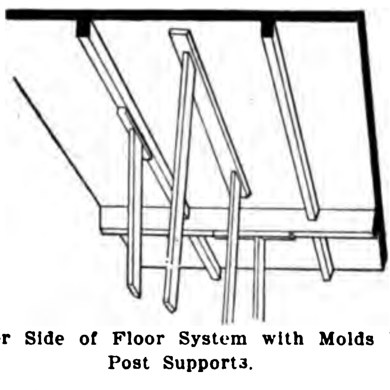


Fig. 435.—Under Side of Floor System with Molds Removed and Post Supports.

should be left in place for three or four weeks, after removing while the beams and girders should be supported by props until the concrete is six or eight weeks old.

Fig. 435 shows the under side of a floor system with the moulds removed, but supported by occasional posts. It is advisable when the floor surface is exposed to the rays of the sun in warm weather to cover it with cloth or burlap, kept continuously wet or kept well sprinkled; otherwise bad shrinkage cracks will be formed.

Finish of Floors.—This will depend on the purpose of the floor. Tiling or mosaic work may be laid on the top surface of the concrete, with a plaster bed, in the usual manner. For ordinary factory floors a finish of rich cement mortar from 1/4 to 1 in. in thickness, trowelled smooth, is sometimes used. If a better finish and a harder surface is desired, a granolithic finish may be used. This, if possible, should be laid while the concrete is still soft. If a top surface of wood is desired, nailing strips are set in the concrete or upon it, with a filling of cinder concrete between them, and the floor surface nailed to them.

The methods to be used in finishing a concrete floor surface are similar in all respects to that used for finishing sidewalks, and fully described on page 127.

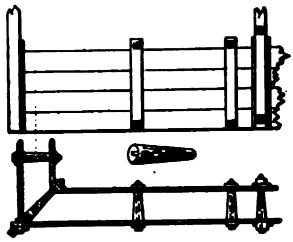


Fig. 436.—Potter's Wall Form.

The Construction of Walls and Partitions.—Forms for walls and partitions may be constructed as in columns, between timber sides extending the entire height of the wall. The reinforcing rods or network are first put in place; the walls of the mould erected, and the concrete, mixed wet, is then put in and rammed with long slender rammers to force it around the metal. Forms of this kind consist of uprights, placed at fairly close intervals, which hold the sheathing in place. Struts and crossties are used to keep the uprights in line and the sheathing at proper intervals apart. Bolts and wire ties, extending through the wall, are used for this purpose. Hennebique sometimes uses a modification of this method. The mould on one side is built in panels all the way up; and the other side is brought up as the concreting proceeds. The longitudinal rods are placed as the concrete is brought up to the proper elevation. This method enables the concrete to be placed in layers of moderate thickness and to be thoroughly rammed as it is put in. The construction of the above type of wall mould consumes a large amount of lumber, which is of little value after being once used. They are frequently used when timber is cheap, as the cost of erection is quite low.

A form of wall mould described by Thomas Potter in his work on "Concrete; Its Use in Building," is shown in Fig. 436. These forms consist of pairs of vertical posts placed at convenient intervals and connected by tie-bolts passing through conical washers to preserve proper spacing between them. Movable frames, consisting of sheathing boards nailed to battens, of the proper length to fit between the upright posts, are put in place and held flush with the inner side of the posts by battens nailed to the side of the posts. Through tie-bolts, also surrounded by conical washers at each batten, maintain the proper thickness of wall between posts. In concreting several sections of sheathing are

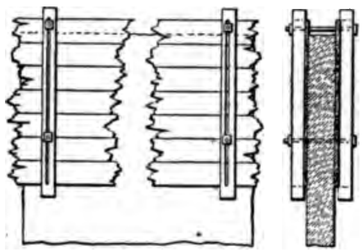


Fig. 437.—Wall Forms Used in Ransome System of Construction.

used, the lower one being removed from the hardened concrete and placed on top of the one last set, and the concrete is then carried on up. To remove a section the bolts are withdrawn, the tapered spacers driven out and the holes filled with mortar. Narrow sheathing boards should be used to prevent warping; they should be surfaced to give the concrete a smooth surface and closely jointed to prevent leaking. In the construction of wall moulds of all kinds it will be well to splay the edges of the boards as done in column moulds, and shown in Fig. 406, page 574.

In the Ransome constructions the sides of the form are carried up as the work proceeds, the planks forming them being held in place by vertical uprights. The uprights, or standards, are slotted, as shown in Fig. 437. These uprights are kept at proper intervals apart by spacing pieces, which are removed as the concrete is brought up. Through-bolts passing through the slots hold the standards firmly against the portion of the wall already built and the spacing pieces placed between them. The sheathing boards are put in position between the standards as the work progresses. When near the top the bolts are loosened in one pair of the standards, pushed up, releasing the lower boards, and the bolts again tightened. When a bolt reaches the lower limit of a slot it is removed and placed at the top of the slot, and so on. The holes left by the withdrawal of the bolts are filled with mortar.

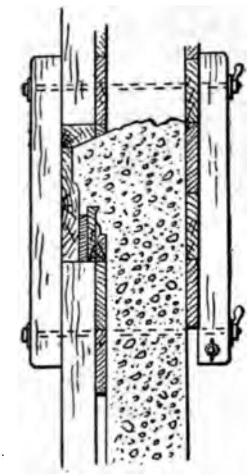


Fig. 438.—Form for Wall Molding.

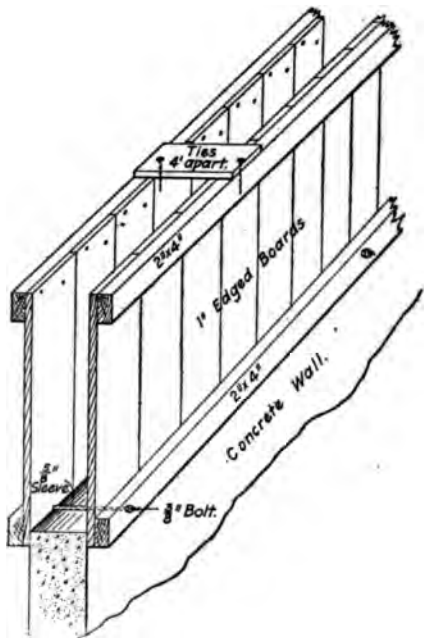


Fig. 439.—Wall Mold for Central Felt and Paper Co.'s Factory, Long Island City, N. Y.

When mouldings in the wall are desired the methods followed are similar to that shown in Fig. 438.

Fig. 439 shows a type of mould similar to the one just described. This mould was used in the construction of a wall for the Central Felt & Paper Co.'s building. Each panel of this mould consisted of two vertical side pieces, 3 ft. high and 16 ft. long. The first course was seated on the foundation and held together by braces. After the concrete had hardened sufficiently the sides were loosened and pushed up until the lower edges lapped slightly on the concrete already laid. In this position they were supported by bolts passing through pasteboard sleeves resting upon the top of the concrete, and their upper edges were held at the proper distance apart by battens nailed to their top edge, about 4 ft. apart. In the use of this type of mould special care must be taken to make a perfect joint between the old and new concrete. Again, great care must be exercised to keep the moulds in line when they are raised, or the wall will be constructed out of line.

When hollow concrete walls are constructed, core boxes, in addition to the usual side moulds, must be employed. These must be made collapsible, so that they can be removed without injuring the concrete. Cross ribs connect the two sides of the wall at frequent intervals, and the whole makes a very rigid wall construction. Collapsible forms similar in design to those shown in Fig. 411, and used for hollow columns, may be used.

When hollow reinforced concrete walls of light construction are to be built, forms like that shown in Fig. 440 may be used. In this form the bolts do not pass through the concrete, but rest upon the top of the core, and the forms are raised when the concrete reaches the level of the bolts. The core boxes are slightly tapered to keep them from slipping down.

Where the wall forms are erected complete before concreting is begun it will be advisable to use a tolerable wet mixture, but where movable panels are used for forms a dry concrete should be used, as it will harden much more quickly than wet concrete, thereby enabling the wall to be carried up more rapidly, and when the cement is tolerably quick setting the operation of concreting may be carried on continuously.

For walls a concrete having a 1 : 2½ : 5 or 1 : 3 : 5 mixture will usually be found satisfactory. The stone should not be greater than ¾ or 1 in. in diameter.

Wall Mould Ties.—Various kinds of ties, some of which have been patented, have been devised to hold the sides of wall moulds together. These are generally better adapted to use in thick walls than in thin walls, such as are used for reinforced concrete work. Bolts with or without sleeves will usually be found satisfactory. These special ties usually consist of the shank of a bolt threaded at both ends, having a special nut or threaded casting at each end. When the form is removed these nuts or castings are screwed off, leaving the bolt in the wall. Another device consists of either ordinary nuts or special castings placed in and near the faces of the wall and connected by wire ties. Bolts passing through the timbers of the frames are screwed into these nuts and tightened by means of nuts and washers on the outer ends of the bolts.

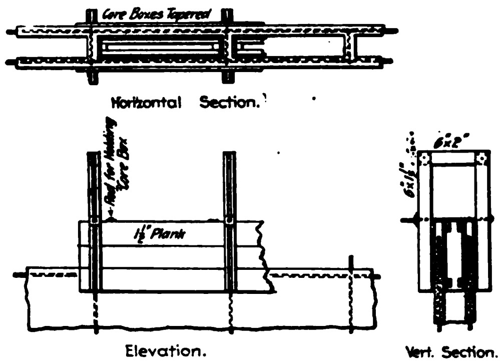


Fig. 440.—Form for Hollow Walls.

When these nuts are loosened the bolts unthread from the buried nuts or castings and the latter are left in the wall. Fig. 441 shows two kinds of wall ties. Through-bolts will generally give less trouble and prove much more satisfactory.

Roof Construction.—The methods used in the construction of

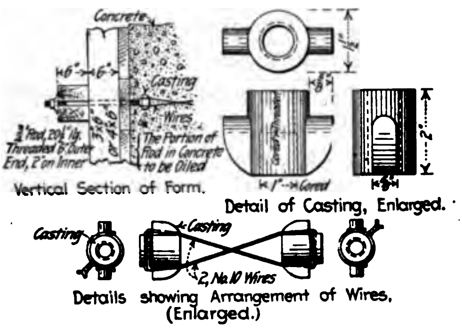


Fig. 441.—Example of Wall Form Ties.

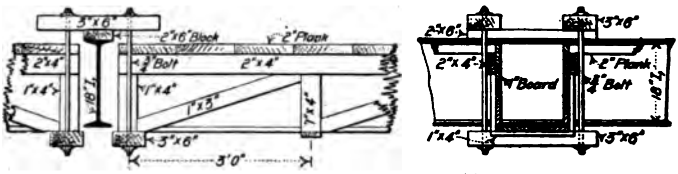
roofs are similar to those used on floors. When the roofs are flat or only have a slight slope, the methods used are identical with those employed for floors.

If the roof has a moderately steep pitch, a dry concrete must be used, as this will stand at quite a steep slope. If a wet concrete is desired, forms similar to those used for walls must be employed.

Concrete roofs may, like floors, be either supported by a framework of steel or may consist of a reinforced slab supported by reinforced concrete rafters and purlins.

A roof constructed of the first kind is described on page 541, and was used for a roundhouse for the Canadian Pacific Railway.

Fig. 442 shows forms used in this construction which consisted of a reinforced concrete roof plate supported by reinforced concrete purlins, spanning between and supported by 18-in. I-beam rafters. The details of the forms are clearly shown in



Part Side Elevation. Cross Section.

Fig. 442.—Form for Slab and Girder Roof.

the sketch. The manner in which they are hung from the top flanges of the beams should be noted.

For steep roofs, domes and other complicated roofs, the moulds are in each case a subject for special study in the draughting room and careful work for the carpenter. In such cases the inner moulds require complicated framing, and when special paneling is desired, or for any other cause outside moulds are needed, they are built up of light timber in panels of convenient widths, or are sometimes made of plaster of Paris, strengthened with wire meshing. Lugs are supplied to keep the moulds at proper distances apart, and ties or bolts are used to hold them together.

When an entire outer section is put in place, the concreting is brought up in the usual manner, a wet mixture being used. After the concrete has hardened the outer moulds are removed and any inequalities filled with cement mortar.

CHAPTER XXV.

RETAINING WALLS.

Retaining walls of reinforced concrete may be used with economy in many localities to replace walls built of cut stone, rubble or concrete. The amount of saving depends, of course, upon the relative price of stone, sand, gravel and cement, and at times may amount to as much as 50 per cent. The cost of forms, as is always the case in reinforced concrete work, will to a large extent determine the amount of saving, and when properly designed should not greatly exceed the cost of forms for a solid concrete wall with a gravity section. It will be found that reducing the sections to a minimum thickness will increase both the cost for forms and for depositing the concrete. Slightly thicker sections, when used, will often prove more economical, as the slight increase in concrete yardage will be offset by the saving on the other items. The usual form of retaining wall used for walls built of brick, stone or concrete is what is called a gravity section, i. e., when the resistance to the overturning of the wall due to its weight counterbalances the overturning moment due to earth pressure against the back of the wall. Reinforced concrete walls do not depend upon the weight of the masonry alone to resist overturning, but substitute for it the weight of the earth resting upon a broad base formed by a slab of reinforced concrete at the heel of the wall, and sometimes a horizontal beam or beams between the buttresses at the back of the wall. The function of these horizontal beams is similar to that of the relieving arches sometimes used with masonry retaining walls.

The economy of the reinforced concrete wall is due to the ability of a wall of thin section to resist the bending moments and shears caused by the earth thrusts when acting as cantilever and simple beams and as slabs supported on two and four sides.

The weight of the earth on and vertically above the reinforced slab forming the heel of the retaining wall, multiplied by the distance of its center of pressure from the toe of the wall, gives the moment of resisting to overturning due to the weight of the earth. This, plus the resistance to overturning due to the weight of the wall itself, gives the total resistance to overturning, and must equal the total overturning moment due to earth pressure multiplied by a suitable factor of security. Sometimes the weight of the wall itself is neglected when computing the resistance to overturning. This is on the side of safety.

The scope of this work precludes a discussion of the methods employed for determining the pressure of earth against the back of retaining walls. This pressure varies according to the nature of the earth, its height above the wall, the amount of moisture it contains, etc. The reader is referred to Merriman's "Walls and Dams," Howe's "Retaining Walls for Earth," Baker's "Masonry Construction," Trautwine's "Engineers' Pocket Book," and other works on the subject for discussions on earth pressure and methods of determining thrust against walls.

In designing reinforced concrete retaining walls provision must be made against sliding. This may be done by providing a wall buried in the earth below the bottom plane of the base, as shown at the toe of the wall in Fig. 446 and at the heel of the wall in Fig. 447.

At times it will be necessary to drive piles, preferably under the toe of the wall, to increase the supporting power of the soil at this point and to anchor the wall against sliding by burying the heads of the piles in the concrete footing. This is shown in Fig. 449, which shows a retaining wall at Seattle, Wash.

Reinforced concrete retaining walls may, in general, be divided into two classes: Walls with an inverted T-section and walls with counterforts.

Inverted T-Section Walls.—These walls are simple in form and easy to construct. They consist, as shown by Fig. 445, page 610, of a thin reinforced vertical wall rigidly attached to a base formed by a reinforced concrete slab. The vertical slab acts as a cantilever beam. The earth pressure increases from zero at the top to a maximum at the upper face of the base, where the bending moment is a maximum. The wall proper is usually increased in thickness from about 6 to 8 ins. at the top to a maximum thickness at the top face of the base. At the latter point there must be sufficient concrete and metal to resist the stresses due to shear and bending moment. The base at the heel acts as a cantilever, and must resist the weight of the superimposed earth resting upon it; while the portion of the base in front of the wall, forming the toe, also acts as a cantilever and resists the upward thrust of the earth caused by the tendency of the wall to overturn about the point of the toe. Hence the base must be reinforced at both top and bottom. Care must be taken that the maximum pressure at the toe does not exceed the safe bearing power of the soil upon which the wall rests.

Having found the moments of the wall at the section where it

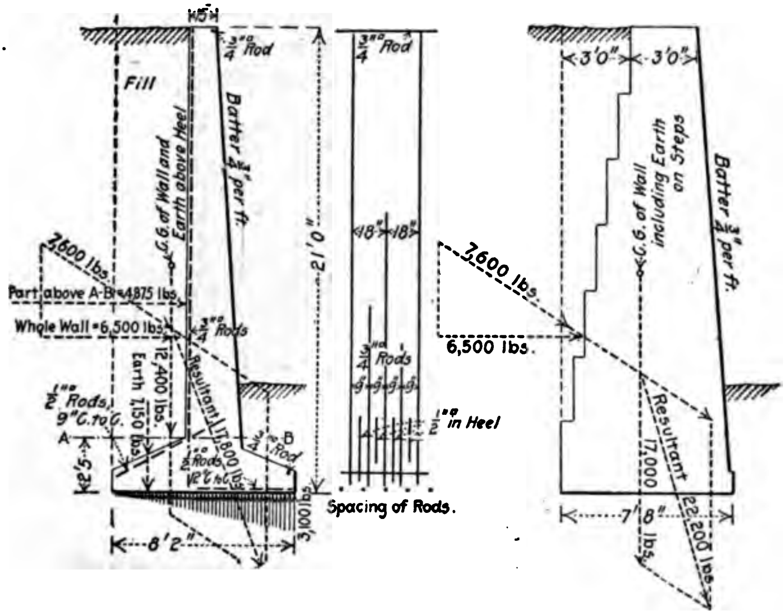


Fig. 443.—Retaining Wall, Lebanon, O.

Fig. 444.—Plan Concrete Retaining Wall.

joins the base, and the moments of the base at the front face of the wall for the toe and the back face for the heel, the sections can be readily proportioned by the equations given in Chapter XIX.

The shear of the wall where it joins the base should be investigated, and if necessary provided for. The reinforcing bars should extend a sufficient length into the base for the adhesion to develop the tensile strength of the bars; or, better still, the tension bars should be firmly attached to longitudinal bars buried in the base. The rods may be bent to hook over the longitudinal or anchor bars or have loops forged on their ends, through which the longitudinal rod can be strung, or they may be attached to the lower reinforcing rods of the base slab.

The width of base necessary will depend upon the angle of repose of the earth, the angle of surcharge, and the weight of the earth. Under usual conditions it will be about 0.5 the height, but may in extreme cases vary from 0.4 to 0.8 of the height of the wall.

Fig. 443 shows an excellent example of a design for a reinforced wall, 21 ft. high, erected at Lebanon, O., by Mr. Frank A. Bone. The diagram of earth pressure is shown in this figure. The size and arrangement of the reinforcements are shown in the

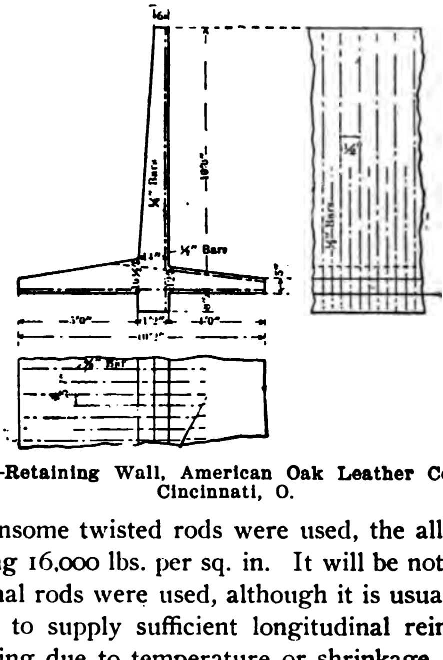


Fig. 445.—Retaining Wall, American Oak Leather Co. Building. Cincinnati, O.

drawing. Ransome twisted rods were used, the allowable working stress being 16,000 lbs. per sq. in. It will be noticed that only two longitudinal rods were used, although it is usually considered good practice to supply sufficient longitudinal reinforcement to prevent cracking due to temperature or shrinkage strains. This wall is 185 ft. long, and cost $13 per lineal foot.

Fig. 444 shows a plain concrete wall of gravity section in the vicinity, built by the same contractors at a cost of $21.20 per lin. ft., or 63 per cent. in excess of the cost of the reinforced wall. The cost of the forms for the above reinforced wall did not exceed that for the plain concrete wall.

Fig. 445 shows a retaining wall of Tee-section used in the construction of a building for the American Oak Leather Co., Cincinnati, O. This wall is 10 ft. 6 ins. high, 6 ins. thick at the top, and has a width of base the same as its height. The arrangement of the reinforcing rods are shown in the drawing.

Walls with Counterforts.—These walls, as shown by Fig. 446, consist of a broad base plate, a thin vertical curtain wall, usually

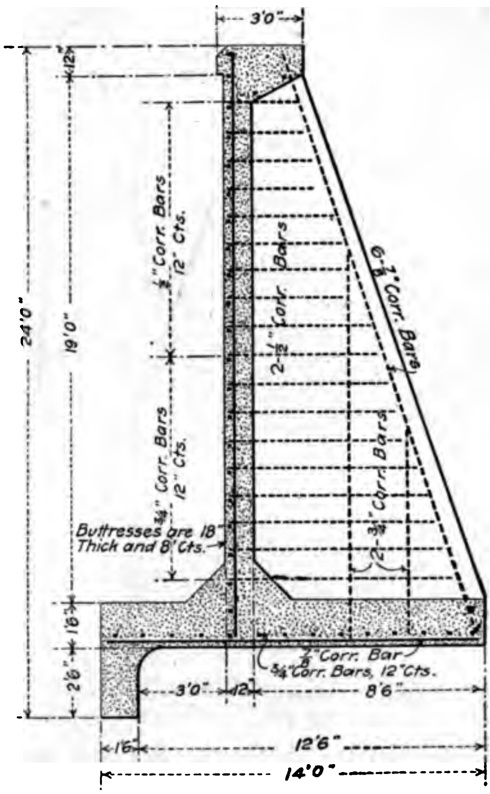


Fig. 446.—Retaining Wall with Counterforts.

varying in thickness from 4 to 6 ins. at the top to 8 or 10 ins. at the bottom, and vertical ribs or counterforts spaced 6 or 8 ft. centers, connecting the base plate and vertical wall. The economy of material for this type of wall is greater than for the wall of T-section, and increases as the height increases, but the cost of the forms is greater. In this type of wall the overturning moment and the bending moment produced by the resultant horizontal thrust at the plane of the top of the base are resisted entirely by the counterforts. The horizontal earth pressure between the counterforts is transmitted by the thin vertical wall slab to the counterforts. When a horizontal beam is used at the top to form a coping, the vertical wall may be figured as a slab supported on four edges. However, in this type of wall, as in the T-section, a minimum thickness of slab will not always prove the most economical.

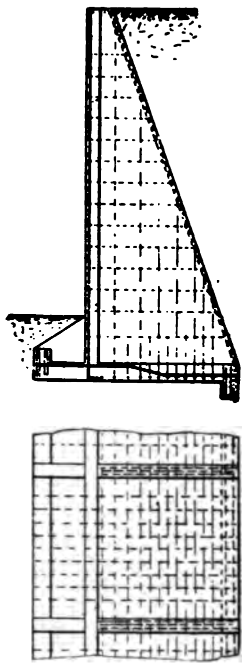


Fig. 447.—Retaining Wall with Counterforts Designed by Hennebique.

The base plate at the back of the wall between the counterforts should be designed as a floor slab, supported by the counterforts, to carry the weight of the earth and superimposed loads coming upon it, while the portion of the base in front of the wall should be designed as a cantilever beam fixed at the wall to resist the reaction of the ground. It should be remembered that the maximum pressure at the toe must not exceed the safe bearing power of the soil. Fig. 446 shows the design of a wall made by the St.

Louis Expanded Metal Co., and Fig. 447 a wall of Hennebique construction.

Sufficient metal must be placed in the back of the counterforts to care for all tensile stresses due to overturning. Horizontal reinforcing rods are placed at frequent intervals throughout the height of the counterfort to tie the face slab to it, while occasional vertical rods will assist in carrying the tensile strains to

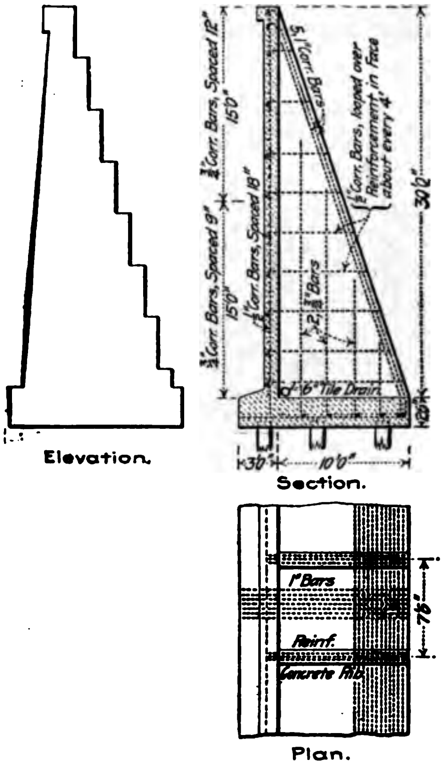


Fig. 448.—Retaining Wall, Great Northern Ry., Seattle, Wash.

the base. The amount of reinforcements necessary for the wall and base slabs and girders is determined and the rods located in the manner usually employed for slabs and beams; this has been explained in Chapter XIX. In addition to the amount of longitudinal reinforcement necessary to care for transverse strains, provision should be made for temperature and shrinkage strains, or when this is not done the wall should be built in sections of about 50 ft., with expansion joints between adjacent sections.

The questions of temperature and shrinkage strains will be discussed further on in this chapter.

A good example of a high reinforced concrete retaining wall of the counterfort type is the wall adopted for the terminal yard at Seattle, Wash., by the Great Northern Railway. This wall supports a street, and varies in height from 2 to 37.8 ft., and will be approximately 2,000 ft. in length. Mr. C. F. Graff, of the engineering staff of the Great Northern Railway, states that a comparison of cost between a plain concrete wall of gravity sec-

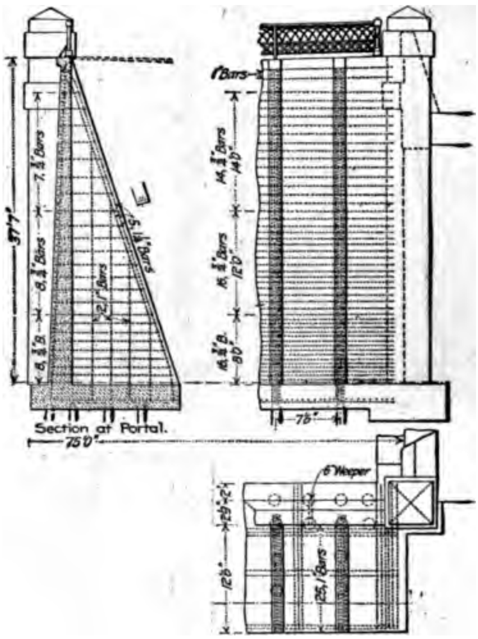


Fig. 449.—Retaining Wall, Great Northern Ry., Seattle, Wash.

tion and a wall of the counterfort type (Fig. 448) gave the following saving for walls of heights varying from 10 to 40 ft., assuming a section of wall 1 ft. long and figuring the amount of steel used at 4½ cts. a pound, evaluated in terms of concrete at $6 per cu. yd., in place:

TABLE LXXII.

Height of Wall. Feet.	Cu. ft. Concrete, Plain Wall.	Cu. ft. Concrete. Reinforced Wall.	Saving, per cent.
10	44	34.9	20.4
20	110	69.9	36.4
30	226	127.8	43.4
40	396.4	218.0	45.0

It was assumed in the above estimate that the extra cost for forms and a higher grade of concrete for a reinforced wall was counterbalanced by the saving in piling necessary for the plain concrete wall.

Fig. 449 shows elevation section and plan of the wall at its highest end where it joins the portal. At the highest point the wall is 37 ft. 7 ins. in height. Fig. 449A shows an elevation and section of the wall where it is 31 ft. 7 ins. in height. The general dimensions and reinforcement employed are shown on the drawing. In computing sections of face and base of wall they were considered as composed of a series of independent beams lying side by side, giving an additional factor of safety, as there is in reality a supported slab action.

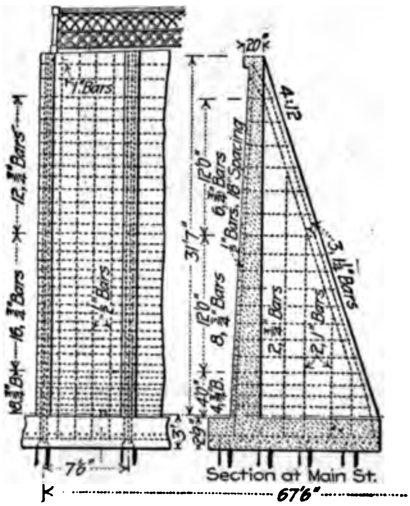


Fig. 449A.—Retaining Wall, Great Northern Ry., Seattle, Wash.

Piles were driven, as shown in the part plan (Fig. 449), to compact the earth, to support the toe of the wall and to prevent the structure from sliding forward. Scaffolding was put up to facilitate the erection of the steel skeleton work. Near the top of this scaffolding the two top 1-in. horizontal face bars were securely fastened in exact line and elevation, and the long diagonal 1¼-in. bars running down the back of each rib were hooked on these and swung into proper position at the bottom. Some of these bars were 42 ft. in length, and were kept from sagging by wooden cross pieces nailed to the falsework. The ½-in. vertical face bars were then hung from the top horizontal ones and held in place in a similar manner. Next the vertical bars in each rib were placed, being stuck in the ground at the bottom and held at the top by wire tied to the scaffolding.

In construction, 3 ins. of concrete was first placed above the

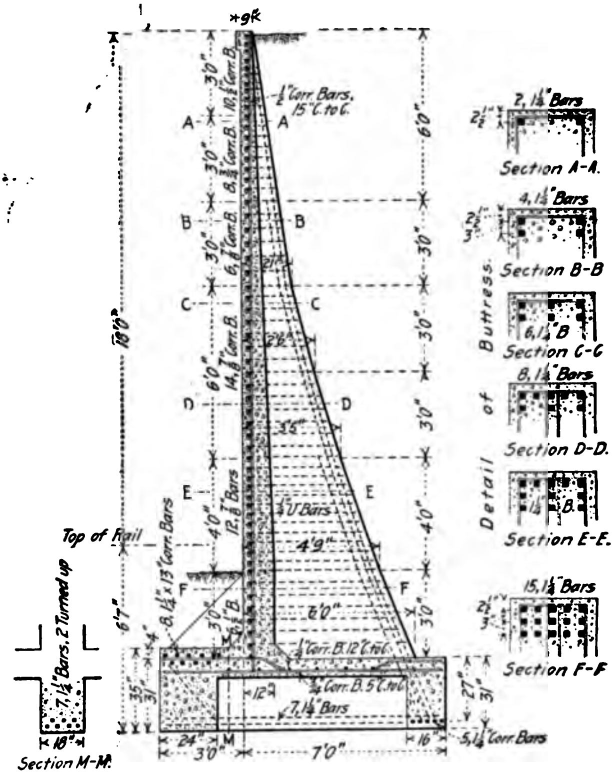


Fig. 450.—Retaining Wall, Brooklyn Grade Crossing Commission.

top of the piles, the horizontal longitudinal rods were put in place, and then the concreting carried up throughout the whole section. As the work was brought up the horizontal bars in the face and ribs were put in place, care being taken in all cases to bed them in fresh concr ; The laps where the rods were spliced were made at the r ,, a 2-ft. lap being used for the base and 1½-ft. lap for the f wall. Corrugated bars were used throughout. A 1:2:4 m of Portland cement, sand and trap-rock was used for the concrete. A fairly wet mixture was employed, being deposited in 6-in. layers and thoroughly tamped.

Fig. 450 shows a design of a proposed retaining wall for the Brooklyn Grade Crossing Commission by H. C. Miller & Co. The reinforcing metal is St. Louis Expanded Metal Co.'s corrugated bars. The total height of this wall is 24 ft. 7 ins., and the buttresses are spaced 15 ft. centers. The dimensions, reinfo :ement and general features of construction are clearly showı · the aw- ere .ving is in

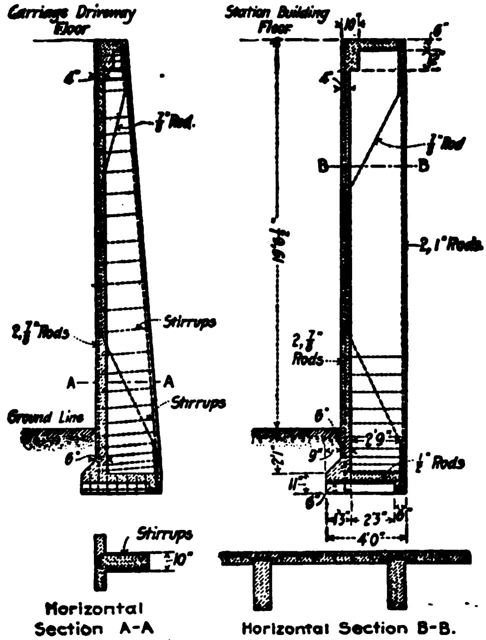


Fig. 451.—Retaining Wall with Counterforts, Atlanta Terminal Station.

drawing. This wall is exceptionally well designed, and uses a minimum amount of material.

In the construction of the Terminal Railway Station at Atlanta, Ga., several modifications of the usual type of wall with counterforts were used. Fig. 451 shows two sections of a wall 194 ft. long and 22 ft. 6½ ins. high. The spacing of the counterforts varies from 4 to 6 ft. On another part of this work the wall shown in plan, elevation and section in ꞔ. 452 was used. Figures 453 and 453A show the details of a ·ll used to protect the end and side of a building from an emb ·ment filling. As will be seen, the usual solid buttress is here ·, ied by a skeleton buttress. The details of construction are clearly shown in Figs. 453 and 453A.

Fig. 454 shows a modification of the usual type of wall with counterforts. This wall is of Hennebique construction, and was used to support the sides of a sunken street near the Gardens of the Trocadero, at the Paris Exposition of 1900. The wall was

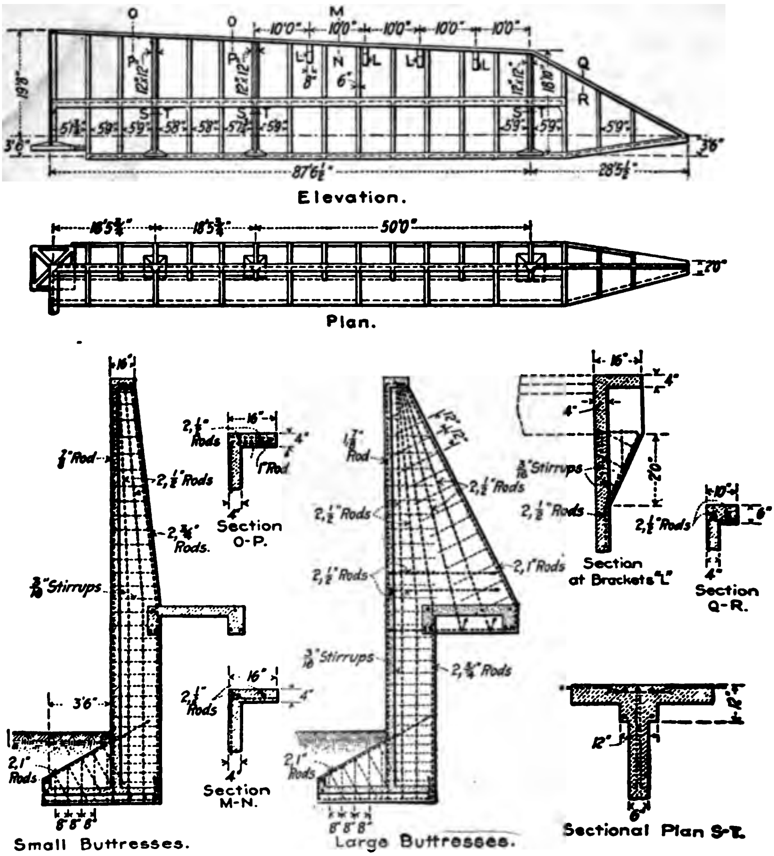


Fig. 452.—Retaining Wall with Counterforts, Atlanta Terminal Station.

built in sections about 20 ft. in length, each section or panel being made up of a facing strengthened at its back by three buttresses. Two horizontal beams connected the facing and buttresses. Shallow buttresses below the street level were used to strengthen the base slab at the toe of the wall. Two separate beams, at different levels, which act as relieving arches, reduce largely the thrust of

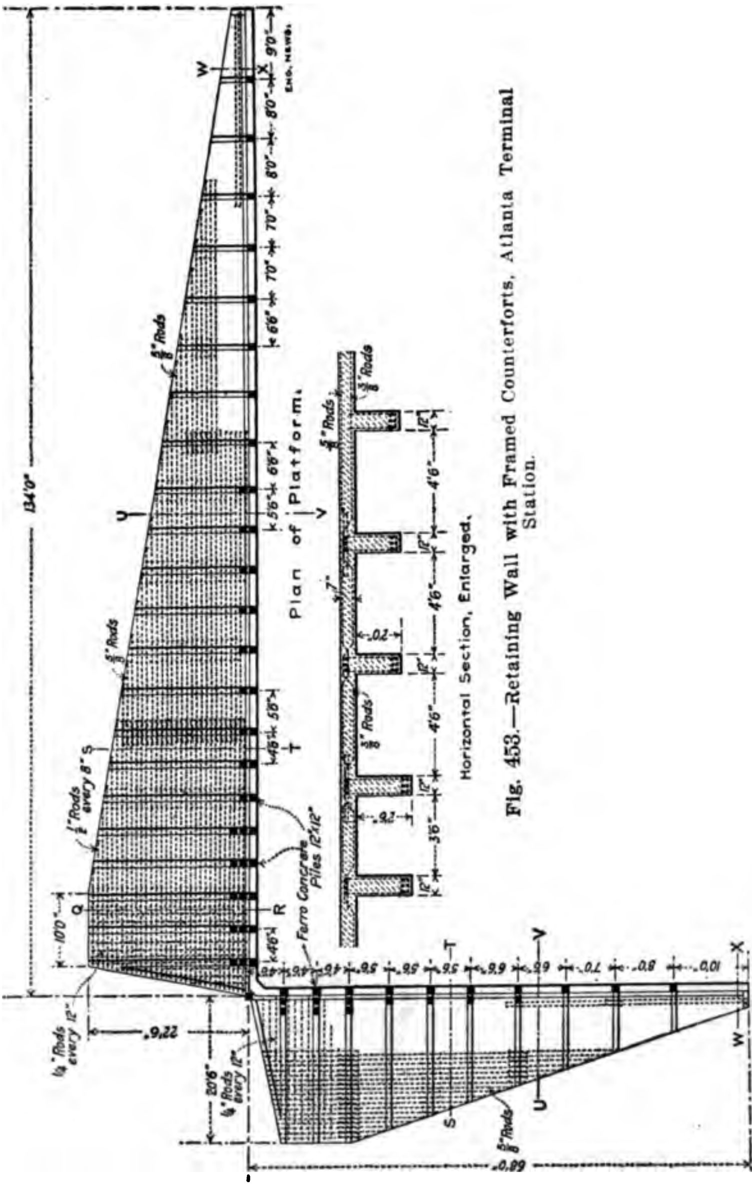


Fig. 453.—Retaining Wall with Framed Counterforts, Atlanta Terminal Station.

the earth upon the vertical face of the wall and assist in sustaining the earth by the weight of the earth coming upon the beams. This arrangement also greatly reduces the amount of excavation necessary in building the wall. The vertical face was reinforced

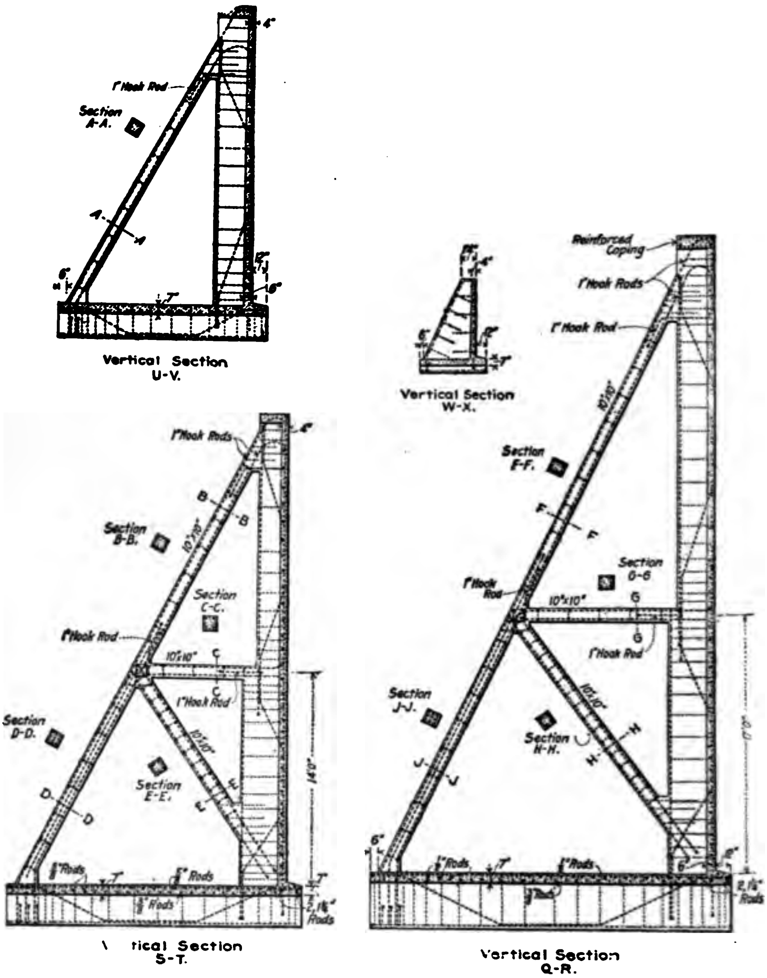


Fig. 453A.—Details of Counterforts Shown in Fig. 453.

with two series of vertical bars, combined with one series of horizontal bars, with increasing spacing toward the top of the wall. The vertical bars were bent over at right angles at the top to support the reinforced coping. The reinforcements of the buttresses consisted of inclined bars tied together by straps and supported by horizontal bars. The horizontal beams had reinforcing bars, run in both directions and spaced about 8 in. centers. The beams were further strengthened by flanges at their edges. The arrangement of the reinforcement and general features of construction are shown in Fig. 454.

Braced Walls.—Reinforced slabs are frequently used for area and cellar walls. When so used they are braced at both top and

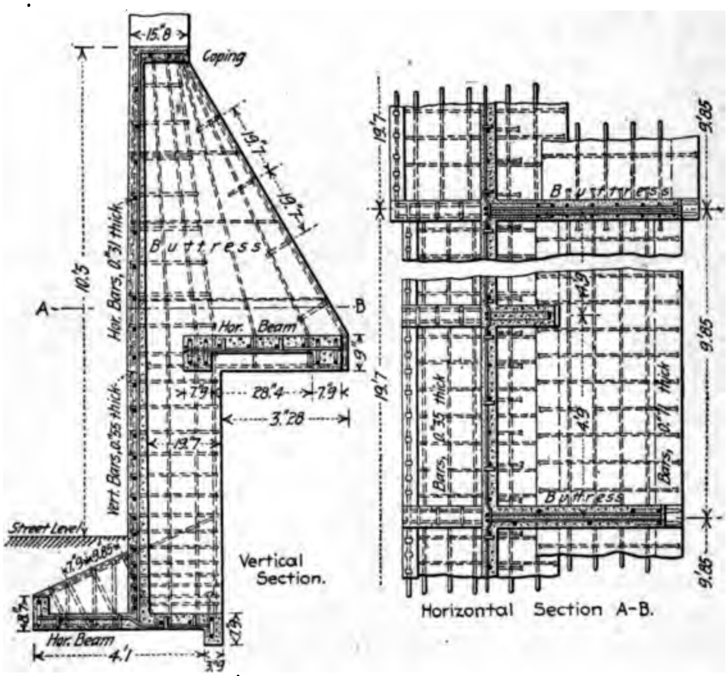


Fig. 454.—Retaining Wall for Sunken Street, Paris, France.

bottom, and may be considered when they are figured as slabs supported at top and bottom and loaded with a uniformly increasing load from top to bottom.

Expansion and Contraction.—Some provision should be made against cracks due to shrinkage and temperature stresses in walls, dams, pipes, sewers, etc. Unless special provision be made, points of failure or cracks will occur at intervals such that the frictional and other resistances to movement of the section between the cracks are greater than the tensile strength of the wall.

Two methods may be used to avoid unsightly shrinkage cracks: First, the wall may be divided into sections of about 50 ft. and provided with expansion joints to allow for movement and to localize any possible cracking; second, sufficient longitudinal reinforcement may be provided to care for all stresses due to expansion and contraction, and cracking will be avoided. The steel, when so used, equalizes the strain between different sections along the length of the structure and causes it to stretch as a homogeneous material, thereby avoiding dangerous local cracks. When steel is properly distributed through the wall it will stretch about ten times as much without visible cracking as when no steel is present.

As already explained, cracks first become visible in the under side of a reinforced concrete beam when the elongation becomes equal to from 0.001 to 0.0013 of its length. The coefficient of expansion of concrete may be taken at 0.0000055. To produce an elongation of 0.001 will require a change of temperature of $\frac{.001}{.0000055} = 182°$ F. The maximum range of temperature under ordinary conditions will not exceed 125° F., and is probably considerably less than this. Assuming 75° as a probable variation under ordinary conditions, the total change of length due to 75° change of temperature will be $0.0000055 \times 75 = 0.0004125$ part of its length. If it be assumed that the wall is fixed at both ends and subjected to a fall of 75° F., the resulting thermal stress in the steel will be equal to that required to stretch the wall 0.0004125 times its length, i. e., 0.0004125 times the modulus of elasticity of the steel. This will give a thermal stress in the steel of $0.0004125 \times 30,000,000 = 12,375$ lbs. per sq. in. If the modulus of elasticity of the concrete is 3,000,000, the thermal stress in the concrete will be $0.0004125 \times 3,000,000 = 1237.5$ lbs. per sq. in., or about three times its ultimate strength. If no reinforcement is used, the concrete will crack.

The maximum stress coming upon the steel when reinforcement is used will be its own thermal stress of 12,375 lbs. plus the strength of the concrete at its elastic limit. This limit may be taken at its ultimate strength.

Assuming an elastic limit of steel at 34,000 lbs. and the ultimate tensile strength of concrete at 300 lbs., and taking a wall with a sectional area of 100 sq. ins., the area of metal required to care for the thermal stress in the concrete will equal $\dfrac{300 \times 100}{34{,}000 - 12{,}375}$ $= 1.39$ sq. ins., or 1.39 per cent.. If steel having an elastic limit of 54,000 lbs. is used, we have $\dfrac{300 \times 100}{54{,}000 - 12{,}375} = 0.75$ per cent.

Stresses in Concrete Due to Setting.—As has been stated, the contraction of concrete when setting in air, or expansion when setting in water, may amount to from 0.0002 to 0.0005 of their length. Assuming that when the concrete sets in the air its shrinkage amounts to 0.00035 part of its length, the tensile stress resulting will amount to $0.00035 \times 3{,}000{,}000 = 1{,}050$ lbs. per sq. in. Unless the concrete is reinforced, cracks will open up along the length of the wall if it is not free to move. The number and amount of the cracks will depend upon the amount and character of friction on the foundation, the strength of the concrete, etc. The greater the friction, the shorter the intervals between the cracks, and the stronger the concrete the farther apart will be the cracks. A thorough distribution of steel reinforcement throughout the wall will prevent cracking, and if the steel is fixed at the end of the wall it cannot change in length, and the summation of the stress on the steel due to the shrinkage of the concrete will become equal to zero. Where the section of the concrete is weakest the steel will be in tension, and where strongest the steel will be in compression, and the algebraic sum of all the stresses will become equal to zero. The maximum stress which may come upon the steel will be equal to the ultimate tensile stress of the concrete, and the maximum compressive stress in the steel will be that due to the total deformation, i. e., $0.00035 \times 30{,}000{,}000 =$ 10,500 lbs. per sq. in. The amount of steel necessary to care for the shrinkage stresses alone, assuming, as before, 300 lbs. ultimate tensile stress in concrete and 34,000 lbs. per sq. in. as the elastic limit, and assuming a wall section of 100 sq. ins. will be, as before $\dfrac{300 \times 100}{34{,}000} = 0.88$ sq. in., or 0.88 per cent.

Thermal and Shrinkage Stresses Combined.—It is probable that the thermal stress in the steel due to its contraction will neutralize the compressive stresses due to the shrinkage of the concrete. It is usually assumed that the maximum tensile stress in the steel can never exceed its own thermal stress, which, under the conditions fixed above, is 12,375 lbs. per sq. in. plus the ultimate strength of the concrete. Otherwise the concrete would fail. Under this supposition, it will not be necessary to take into account the tensile stress due to shrinkage. In any event, by keeping the wall wet during the setting of the cement, all danger due to shrinkage strains may be avoided and thermal stresses only need to be provided for by reinforcement.

CHAPTER XXVI.

DAMS.

Reinforced concrete is particularly adapted to the construction of dams. When so used there is a great saving in material, and on this account a reduction in cost of, in some cases, as much as 20 per cent. Again the space under the apron may be utilized for storage or power house purposes, as for the location of turbines, electric generators, etc. Another advantage is that of securing a practically impervious curtain face wall, without any of the dangerous leaks so troublesome to locate in some masonry structures. If sufficient number of reinforcing rods are used and run in every direction there will be little or no danger of cracking in the deck concrete.

The usual type of reinforced concrete dam consists of an inclined slab of reinforced concrete extending from the heel to the crest, and spanning between and supported by transverse buttresses of concrete, resting upon the foundation. Another inclined slab may or may not be used to form an apron or spillway.

The deck is usually increased in thickness from the crest to the heel on account of the increase in pressure as the water deepens. Sometimes a greater economy in material will result if a design consisting of longitudinal beams spaced closer together toward the bottom, be used to span between the buttresses and support a thin reinforced deck slab spanning between the beams. Less material will be used, but the cost of forms for this kind of structure is considerably greater than for the first type. An objection to this type is the thin deck slab used, which is liable to be injured by the pounding of floating ice or driftwood during floods. Figure 455 shows a design of such a dam. A dam of this type may prove economical under certain conditions, as in tropical countries where the cost of cement is high.

The principles governing the design of reinforced concrete dams are the same as those used for the design of masonry dams. These are fully explained in such text books as Wegmann's

"Design and Construction of Dams," Baker's "Masonry Construction," etc. However, as reinforced concrete dams are usually of triangular cross-section, they have a much wider base than masonry structures, which greatly increases their resistance to overturning. This resistance is further increased by the weight of the water above the face or deck, which usually has an inclination of from 30° to 45° with the horizontal. An increase in the height of the water flowing over a masonry or solid dam increases the pressure thereon and causes the line of pressure to rise, thereby greatly increasing the overturning moment on the dam without in any way increasing the resisting moment to the same. In a triangular dam, with a broad base, sometimes called a gravity dam, as in hollow reinforced concrete dams, when the head of water flowing over the dam is increased, the lines of pressure become more and more nearly vertical, the overturning moment is actually reduced, and the stability is in no way endangered. Until recently, wooden dams have been the only gravity dams used, but their short life and the increasing cost of lumber now, from the standpoint of economy, in most cases forbid their use.

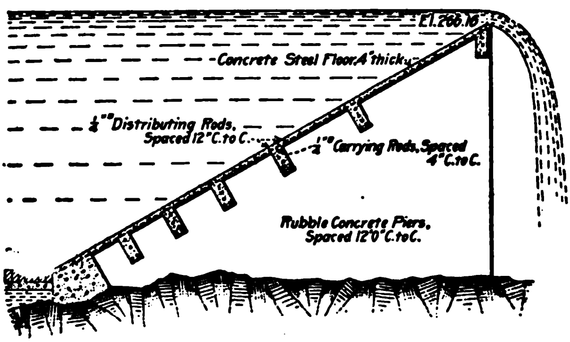


Fig. 455.—Open Front Type Dam.

For the methods of determining the pressure on the face or deck of an inclined dam, together with the center of pressure, see Chapter II. of Merriman's Hydraulics. Again, hollow dams have only a fraction of the weight of solid masonry dams, and hence possess much less resistance to overturning due to the weight of the dam than all masonry structures.

The reduction in weight must also be considered when computing the resistance to sliding. In some cases it will be advisable to fill the hollow dam with sand, gravel, earth or a meager concrete. A 1 : 12 or 1 : 15 mixture will answer the purpose. In the latter event the dam and filling may be so constructed that they will act as a solid mass.

After obtaining the pressure at various depths the problem of determining the thickness of slab needed, and, when used, the size of beams becomes simply a matter of determining the strength of a slab or beam acting under a uniform load. The methods used are the same as those explained in connection with the theory of slabs and beams.

The great reduction in weight and the broad base obtained by the use of this type of structure peculiarly fits it for situations

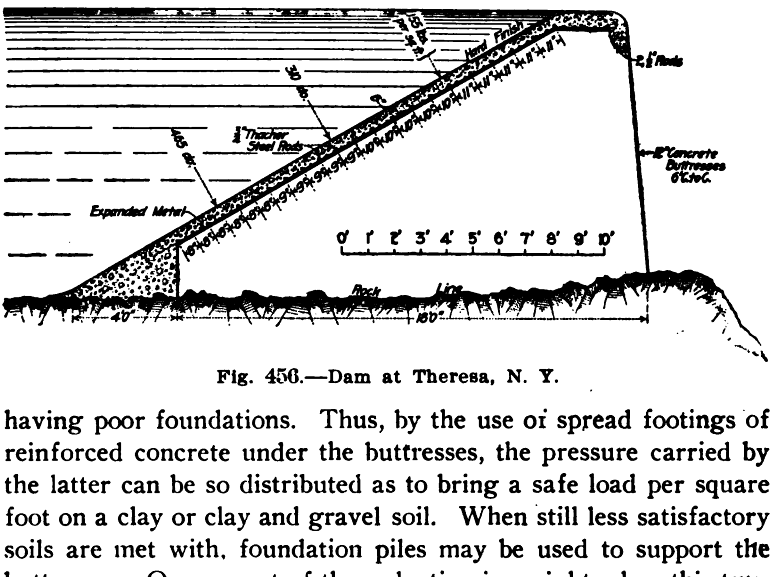


Fig. 456.—Dam at Theresa, N. Y.

having poor foundations. Thus, by the use of spread footings of reinforced concrete under the buttresses, the pressure carried by the latter can be so distributed as to bring a safe load per square foot on a clay or clay and gravel soil. When still less satisfactory soils are met with, foundation piles may be used to support the buttresses. On account of the reduction in weight when this type of structure is used, fewer piles will be necessary than when a solid masonry dam is used, and a great reduction in cost secured. This item alone may at times warrant the adoption of a reinforced concrete dam.

When the foundation is on rock, no provision is necessary against sliding, if the surface of the rock is rough. If it is smooth it is customary to anchor the buttresses to the rock by dowel pins or anchor bolts, or sometimes a heavy cut-off wall, sunk in the rock at the heel of the dam, serves to anchor it and to cut off leakage underneath the dam.

On foundations other than rock, walls buried in the earth and anchored to the buttresses, may be used to prevent sliding, or a few piles having their heads buried in the concrete of the buttresses will suffice to hold the structure securely in place. Where a pile foundation is used, no danger of sliding will result. Sometimes the cut-off wall at the heel or toe may be relied upon alone to prevent sliding.

Shrinkage and temperature cracks may be prevented by the use of sufficient reinforcement. The methods of obtaining the proper sections for the steel have been explained in connection with retaining walls.

The particular form of reinforced concrete gravity dam to be

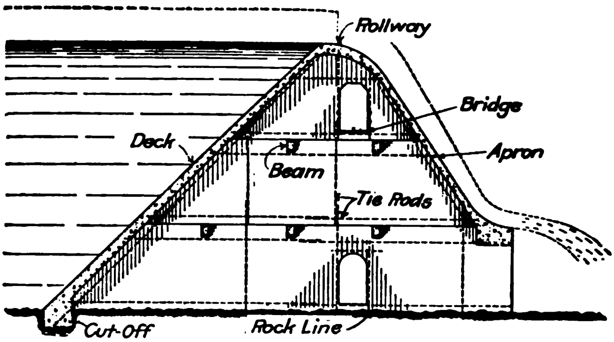


Fig. 457.—Half Open Type of Dam.

adopted will depend largely upon local conditions. The open front dam shown in Fig. 456 is used for dams having a moderate height when located on a ledge of rock of sufficient hardness to resist the erosive action of the overflow of water and ice.

A slight modification of this type consists of extending the slab or apron a slight distance beyond the crest to pitch the ice and water far downstream, away from the foot of the dam.

If the apron is carried down to within 6 or 8 ft. of the base and curved to the bucket form shown in Fig. 457, the half-apron dam is secured. This form of bucket near the toe gives the water a high initial velocity in a horizontal direction and discharges it far below the toe of the dam.

When both the back and front of the dam are covered, what is known as the curtain dam is obtained. (See Fig. 458.) The pitch of the apron is generally easier than that given to the apron of a solid dam. Vents are placed in the apron just below the crest, for the purpose of admitting air behind the sheet of water to kill the partial vacuum which would otherwise form under high velocities of overflow during flood, and which is the cause of the "trembling" of dams.

If the dam is high, and the foundation on ledge rock, the floor shown in Fig. 458, as well as the cut-off wall at the toe, are omitted. When the foundation is on cemented sand, clay, or hard-pan, sheet piling is driven to a sufficient depth at the heel and toe of the dam to insure tightness by puddling, and the concrete placed over and about the head of the piling. Draught holes are placed in the toe to carry off all seepage. Weep holes may be placed in the floor to prevent upward pressure on the floor. This

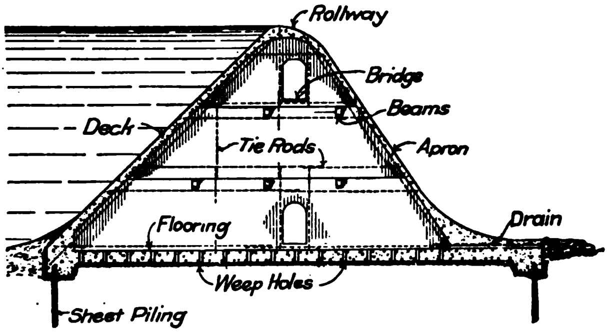


Fig. 458.—Curtain Type of Dam.

arrangement of the foundation prevents any upward pressure from within or below which might endanger the safety of the dam.

A modification of the type shown in Fig. 456 is the form shown in Fig. 459. This type is designed for low heads on alluvial and clay foundations. The floor takes up the effect of the falling water and delivers the water parallel to the bed of the stream and prevents gouging out the bed of the stream below the dam. The sheet piling at heel and toe act as a cut-off and help anchor the dam. The light weight, together with the broad base of a reinforced dam of this type greatly reduces the pressure upon the subsoil, thereby enabling such a dam to be used in locations where the weight of a solid dam would make its use prohibitive. The dam shown in Fig. 459 was designed for a 5-ft. head on a New

Jersey stream, where the foundation is strictly mud, without a trace of rock or gravel. The concrete floor is reinforced in both directions and on both edges, so that the whole dam acts as a unit. The inclined deck is somewhat steeper than generally used, and is braced by buttresses in the usual manner. The overflow is taken directly on the floor of the dam.

The dam built in the Fall of 1904 at Fenelon Falls, Ont., is a good example of a reinforced concrete dam of moderate height. This dam is 10 ft, high and 194 ft. long, and rests upon a smooth limestone ledge. It was built complete in the remarkably short period of 22 ordinary working days. The dam consists of triangular buttresses 12 ins. in thickness, spaced 10 ft. centers and anchored by steel dowels to the ledge, supporting an in-

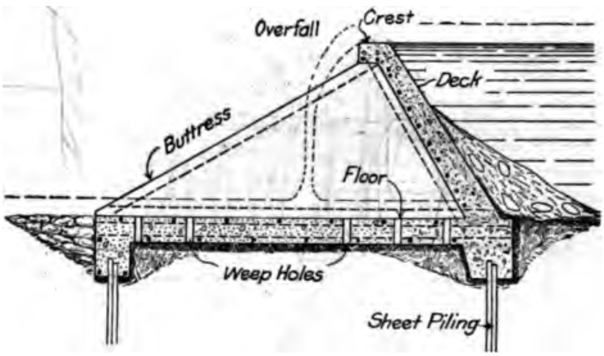


Fig. 459.—Open Front Dam with Floor and Buttresses for Alluvial Streams.

clined deck varying in thickness from 7 ins. at the top to 10 ins. at the bottom, where it increases in thickness to form a cut-off wall. Figure 460 shows a cross-section of the dam.

No reinforcement is used in the buttresses. The deck is reinforced with ⅞ and ¾-in. Thacher bars, spaced from 6 to 7 ins. centers, extending between the buttresses. A secondary reinforcement of fence wire is provided to prevent hair cracks. The deck is figured for a factor of safety of 5, using Thacher's formulas. The maximum load coming upon the buttresses is 4.5 tons per sq. ft.

This dam forms a portion of a horseshoe dam, the water flowing past the face of the dam. To protect the buttresses from drift ice and logs, a reinforced concrete facing slab, shown in the drawing, was provided along the back face of the buttresses.

The dam was completed Nov. 11, 1904, and the full head of water turned on 18 days after the last concrete was laid. This dam was designed and built by the Ambursen Hydraulic Construction Co., Boston, Mass.

Another small reinforced concrete dam, which was among the first to be constructed, was built for a small water power plant at Theresa, N. Y. This dam is 11 ft. high, 120 ft. long and was built on solid rock. Concrete buttresses, 12 ins. thick, spaced 6 ft. centers and anchored to the rock foundation with 1¼-in. anchor bolts 3 ft. long, support a concrete slab reinforced with ¾-in. Thacher bars and expanded metal. The crest is strengthened by a 6 x 8-in. reinforced beam. Figure 456 shows a transverse section, with general dimensions, spacing of rods, etc. A

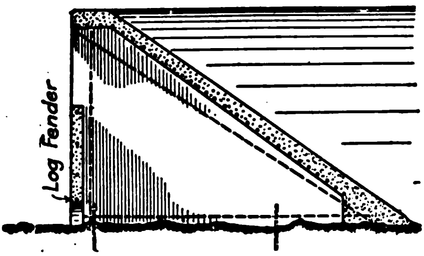


Fig. 400.—Dam at Fenelon Falls, Ontario.

concrete composed of 1 part Portland cement, 2 parts sand and 4 parts broken limestone, was used for the deck slab, and a 1:3:6 mixture for the heel and buttresses. It is stated that about 125 cu. yds. of concrete was required for this dam.

The reinforced concrete dam constructed for the American Wood Board Co., on the Batten Kil River, near Schuylerville, N. Y., is a good example of a hollow reinforced concrete dam of somewhat greater height than those already described. The crest of this dam has an average height of 25 ft. above the bed of the river and a maximum height in the channel of 28 ft., with a length of 250 ft. between abutments. The foundation is of Hudson River shale, moderately firm in texture, but not strong enough to withstand erosion due to sheer overflow. Figure 461 shows a cross-section, a partial front view and partial longitudinal section on the center line.

The general dimensions, details and reinforcing material used are given on these sections.

The form of dam used is such that when its weight was calculated and combined with the water pressure the resultant lines of pressure were found to be more nearly vertical as they approach the crest of the dam, and in all cases to fall well within the base,

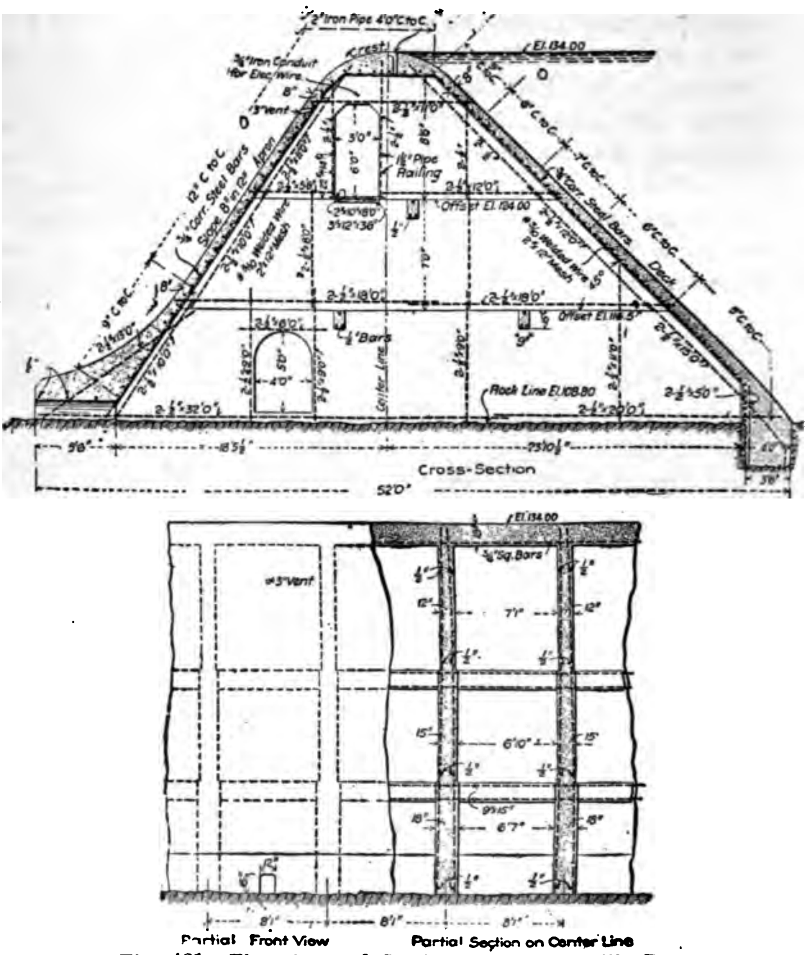


Fig. 461.—Elevation and Sections of Schuylerville Dam.

assuring the absolute stability of the structure under any flood. The angles of these resultants were greater than the angle of friction between the dam and its base, thereby doing away with any possibility of sliding. The buttresses in this dam are 8 ft. 1 in. centers, and have a thickness of 12, 15 and 18 ins. from the top downward in three benches of approximately equal height.

Reinforcing rods were placed vertically, horizontally and along the edges, as shown in the drawings. The concrete used for the buttresses was a 1:3:6 mixture, the stone being 2½ ins. and under in size. The maximum load coming upon the buttresses at any point is 5 tons per sq. ft. No excavation was done for the buttresses, but the rock was thoroughly cleaned off by means of a water jet before depositing the concrete. A trench about 3 ft. wide and deep receives the solid concrete cut-off wall forming the heel of the dam. Three longitudinal beams of reinforced concrete extend between the buttresses, adding greatly to the stability of the structure. These beams are shown in both the elevation and cross sections. The crest is 24 ins. thick in the thickest part, and it was planned to contain sockets for flash boards, but these, through an oversight, were omitted. The deck or face is 9 ins. thick immediately below the crest, and increases to 11 ins. at the bottom. The apron has a uniform thickness of 8 ins. from the crest to the curve of the toe, where the thickness of the toe concrete increases.

Johnson corrugated bars, spaced as shown on the drawing, were used for reinforcement. The sizes and spacing of the bars for the deck were so proportioned as to give a factor of safety of 5 with concrete six months old when the crest is under a 5-ft. flood. Underneath the bars and supporting them is a netting of wire mesh. The deck and apron were laid of 1:2:4 concrete, with aggregates below ¾-in., using as wet a mixture as possible. No special finish was given to the surface, and when the dam was filled no perceptible leak could be found.

The rock at the site of the dam slopes slightly toward its toe and drain openings were provided by which its drainage is assured at ordinary stages of water. During the flood, however, the height of the water is always the same inside the dam as below it, there being free communication through the drain openings. The shale foundation is seamed with minute cracks, which in the case of a solid dam might allow sufficient leakage to create an upward pressure on the base of the dam and endanger its stability. The possible leakage can cause no trouble, for as soon as the water appears below the cut-off wall it is drained off. The 3-in. vent openings shown below the crest in each bay serve to admit air behind the over-fall, thus breaking the partial vacuum and preventing the trembling of the dam. Openings are provided through the buttresses at the top and bottom to allow easy access to all bays. A bridgeway through the top openings affords a passageway through the dam underneath the crest. This footway, which is 16 ft. above the river bed, is dry and well ventilated, is lit by an incandescent lamp in each bay and is used as a passageway from the mill on the north bank to the railway station on the south bank of the river.

In the construction of this dam cofferdams were used, which inclosed all but about 70 ft. of the site near the south end, which was left for a by-pass. The portion of the dam within the cofferdam was all completed except a portion of the deck and apron walls in the lower part of six bays near the middle of the river, which were left for sluiceways when the dam site at the south shore was shut off by a second cofferdam. After the south portion was completed and the cofferdam was removed the sluiceways were closed.

The method of closing the sluiceways is unique. The deck and buttresses of the six bays forming the sluiceways were extended, as shown in plan and vertical section in Fig. 462, to form a horizontal shelf through which the openings forming the sluiceways were left.

Grooves were formed in the buttresses and in the concrete footing. Rough gates of 6-in. timbers were prepared for temporarily closing the bays, and two loose-fitting forms, as shown in the figure, were provided for the construction of the permanent wall of the open bays. A lattice work built up in advance by wiring together horizontal and vertical reinforcing bars of sufficient length to bear on the concrete at the sides and top was used for the wall reinforcement. When everything was ready one of the openings was closed by placing the timber gate in place, the forms and reinforcement put in place, and the concrete deposited. A second opening was closed in the same manner, and lastly the four remaining openings closed at the same time and in a similar manner. Soft coal ashes were dumped in the stream above to close any temporary leaks in the wooden gates. Two 3-in. pipe drains were set in holes previously prepared in the bottom edge of each form to carry away any water leaking past the gates, thereby preventing damage to the green concrete. When the concrete was sufficiently hard the holes were closed with two soft pine plugs, enclosing a plug of concrete. Flash boards were set to divert the water from the openings in the apron, and under this protection forms were set and the concrete put in place.

Figure 463 shows a design of device for holding a flash-board on hollow concrete dams, but, on account of an oversight, omitted when the Schuylerville dam was constructed. The rod is operated from within the dam; by reversing the hook the rod

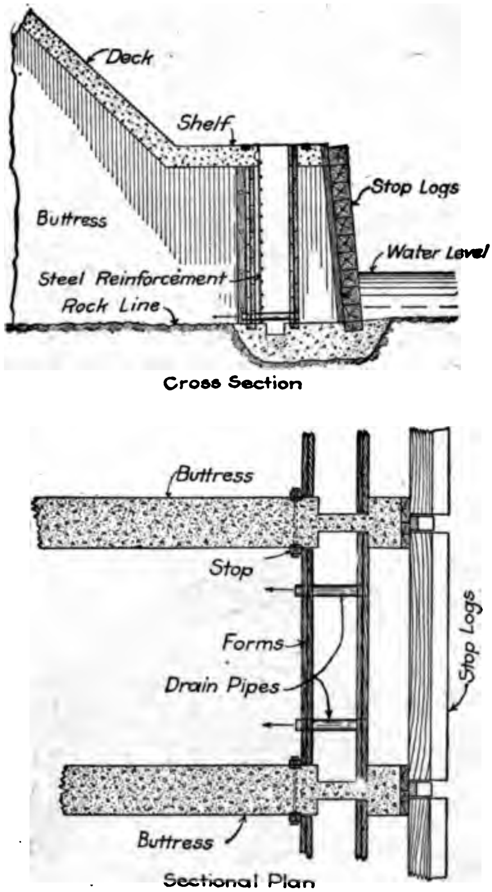


Fig. 462.—Details of Schuylerville Dam.

can be drawn down and the hook will rest in the socket and the boards will then float away. An inverse operation enables the boards to be placed in position.

Mr. Geo. F. Hardy, of New York City, was consulting engineer, and Tucker & Vinton general contractors for this structure. The author is indebted to the latter for information in regard to this dam.

Danville Dam.—Figure 464 shows the method used for increasing the height of an old masonry dam at Danville, Ky. The ma-

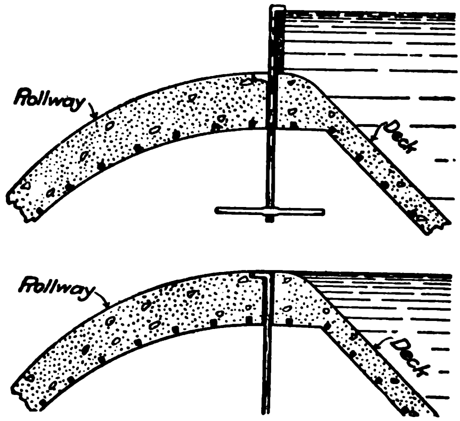


Fig. 463.—Scheme for Flashboards on Reinforced Concrete Dams.

sonry section had already been strengthened by placing a heavy bank of quarry gravel and clay above it. As it was desired to raise the height of the dam 4 ft., the buttresses and reinforced slab construction shown in the figure was adopted. Openings

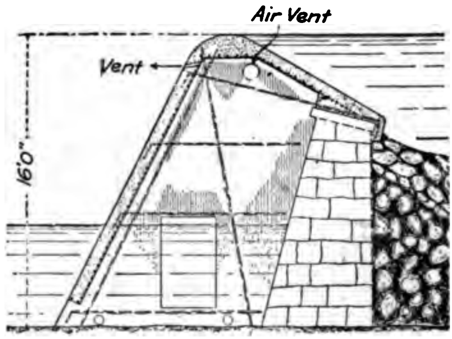


Fig. 464.—Dam at Danville, Ky.

were left below the apron to equalize the tail water, and an air inlet is provided by a 6-in. pipe carried out through the abutment and extending above high water mark.

The open space within a hollow dam may be utilized for the location of machinery, as turbines, electric generators, etc. While if the machinery operating gates, flash boards, etc., are located within the dam, access may be always readily had to them.

Figure 465 shows the cross-section of a dam designed by

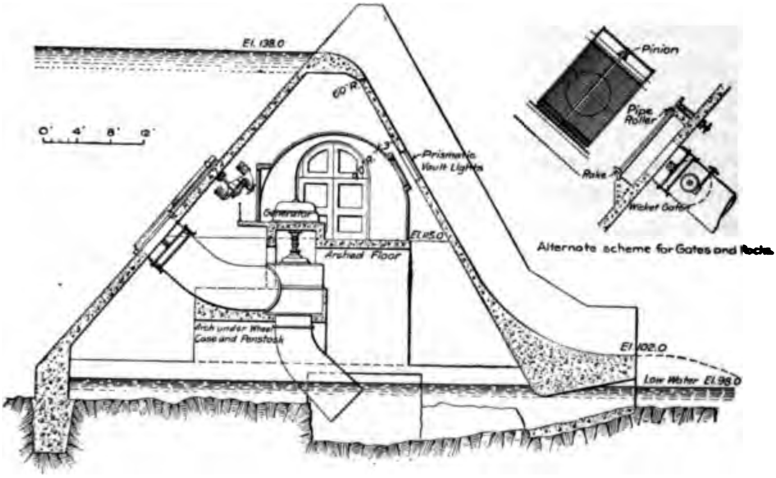


Fig. 465.—Dam at Cannon Falls, Me.

the Ambursen Hydraulic Construction Co. for the Cannon Electric Power Co., Cannon Falls, Minn. As will be noted, the penstock, turbines, generators, operating machinery for gates and tail race are all located within the body of the dam.

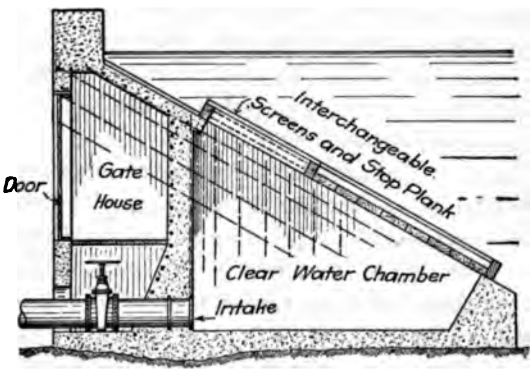


Fig. 466.—Intake and Gate House for Dam at Walton, N. H.

Figure 466 shows an intake and gate house used on a dam constructed at Walton, N. H. As will be seen, the gate house is located all within and in one panel of the dam.

CHAPTER XXVII.

CONDUITS AND SEWERS.

Sewers and water conduits may be built of both plain and reinforced concrete in sizes ranging from a few inches to many feet in diameter. This material has been used much more extensively in Europe for aqueducts and sewers than in this country. Two types of construction are employed, viz.: pipes moulded in advance and laid in much the same manner as cast iron pipes, and conduits moulded in place. American practice has been almost exclusively confined to the latter type of construction, although the first mentioned type has been employed in a few cases, and will undoubtedly have wider use in the future. The use of reinforced concrete in place of plain concrete, brick, vitrified pipe, or metal pipe, is dictated entirely by economy and increased stability.

Pipe moulded in advance is used in Europe from a few inches in diameter up to 6½ ft. and sometimes 7½ ft. in diameter, and 10 or 12 ft. in length.

Monolithic conduits constructed in place possess many advantages, and it is probable that only the smaller sizes of pipes moulded in advance will become popular, as it is impossible to devise satisfactory forms for building conduits in place of a diameter less than about 3 ft.

Much less material is needed to construct pipes moulded before being laid than those moulded in place, as the thickness of the shell rarely exceeds 2½ to 3½ ins., and pipes up to 9 or 10 ins. are usually from ¾ in. to 2 ins. in thickness, while 6 inches is about the thinnest shell that can be successfully used in conduits constructed in place.

Sewers constructed of concrete have been built for from 50 to 70 per cent. of the cost of brick sewers of the same size. When reinforced concrete is used the added cost of the metal is counterbalanced by the greater strength and the saving in concrete, which at times is as much as 50 per cent. This construction should prove under almost all circumstances as cheap as plain concrete sewers.

Reinforced concrete, besides being a cheap material, possesses other advantages for use in conduits, some of which are as follows: It is stronger than either brick or plain concrete, and may be used in lighter sections. It may be strongly reinforced along the length of the pipe and used where there is danger of settlement, without the heavy and expensive foundations usually necessary in such cases. The hardness and smoothness of the surface obtainable with concrete reduces the friction to a minimum and renders it less liable to erosion than other materials.

Concrete sewers built at Duluth, Minn., show very little wear after twenty years' use. Conduits may be made practically water-tight under ordinary heads with little trouble, and when especial care is taken will withstand heads of from 50 to 75 ft. Some twenty to twenty-five years ago a number of concrete sewers were constructed in this country but did not prove very satisfactory, owing probably to the inferior quality of the cement of that time. The great improvement in the quality of cement now used will, however, probably enable concrete to successfully withstand any wear that may come upon it, especially if a granolithic lining be used for a wearing surface.

Some engineers fear concrete will not stand attrition due to possible gravel or small stone carried by water at high velocity, and line their sewers when of large diameter with vitrified brick, and when of small diameter with sections of vitrified clay pipe.

Better workmanship is obtained in constructing reinforced concrete sewers than in unreinforced sewers.

Concrete sewers are usually built in monolithic lengths of not over 50 ft. to avoid shrinkage cracks, but if the section of the longitudinal reinforcement be designed to care for shrinkage and temperature stresses, monoliths of any desired length may be built.

The amount of steel required to care for these stresses may be determined as explained on page 621 in connection with retaining walls.

There are no fixed rules for determining the stresses, sections and reinforcement in conduits under the uncertain loadings due to earth pressure and possible superimposed loads.

The maximum moving load, together with the weight of the material that may come upon the roof of the conduit, should be taken as the maximum loading, although the actual load in many cases will be less than this. In case the fill over the top of the conduit is small, an addition should be made to the moving load for impact.

Whether the roof should be considered as an arch or slab in figuring the stress will depend upon the form of cross section used and the supporting material. In any case, the judgment of the designer will have to be depended upon for the proper treatment of the problem under the existing conditions.

The stresses in the sides of conduits under internal pressure must be entirely taken up by the circumferential reinforcement. The manner of determining the stresses and sections of metal necessary to care for them is as follows:

Let p be the intensity of the internal pressure in pounds per sq. in.
d the internal diameter of the conduit in inches.
T the tension in the shell per lineal in.

$T = \frac{pd}{2}$. If f_s is the allowable working stress per sq. in. in steel, the area of the steel required for each longitudinal foot of the concrete will be

$$A_s = \frac{12\,T}{f_s}.$$

or,

$$A_s = \frac{6\,pd}{f_s}.$$

The reinforcement for sewers and conduits may consist of circumferential rods in the form of hoops or spirals. When hoops are used they should be welded or fully spliced. The rods of adjacent spirals should also be spliced. Longitudinal rods either inside or outside the hooping, depending upon whether the pressure is from within or without, and wired to the hooping at intersecting points, are usually employed. These act as distribution rods, take up shrinkage and temperature strains, and strengthen the pipe longitudinally. When cross strains come upon the pipe provision must be made for an ample section in the longitudinal reinforcement. Expanded metal and various forms of Monier netting are often used for reinforcements in pipe construction.

Pipe Cast in Advance for Conduits and Sewers.—The successful manufacture and use in Europe of reinforced pipe cast in short lengths would indicate that the attention of American engineers should be called to the subject, as undoubtedly there are many cases in which it may also be economically used in this country. The author only knows of two firms in this country who make concrete pipe.

In Europe pipes are generally manufactured at special plants, where the apparatus and process used are controlled by patents. While it is undoubtedly true that the cost of manufacture is thereby reduced to a minimum, it is not essential that such apparatus be used, as pipes can be constructed by the use of simple methods at a cost considerably lower than vitrified clay pipe, cast iron pipe, or concrete pipe cast in place.

Monier Pipes.—The Monier reinforcements for pipes are formed of longitudinal and spiral rods tied together at their intersections. The rods used vary in size from 3/8 in. to 1 in., and are usually spaced evenly. The pipes are molded vertically and are of sufficient thickness to permit ramming. A collapsible core and outside casing is used to form the mould.

If the shell is too thin to be molded in this way, it is formed in the following manner: The reinforcement is placed on a collapsible core of the size of the internal diameter of the pipe. The concrete is mixed stiff and thrown hard against the core, and passing through the reinforcement forms a layer behind the net work, which is shaken during the process. When the first layer is partly set another layer is added in the same manner, but the net work is not again shaken. This process is continued until a proper thickness is secured. The successive layers are about 3/8 of an in. in thickness. The inside and outside is then finished with thin layers of mortar floated on.

Wayss & Co., of Berlin, Germany, employ special molding machinery. The pipes are formed on a rotating drum of sheet iron or wood covered with zinc. The mortar is applied to the drum through a hopper at its upper surface, and the mortar is spread evenly over the drum by rollers. A special arrangement prevents the mortar from breaking away from the lower part of the drum during rotation. After one coat has been put on, the reinforcing network is wound around it and another layer of mortar added and the process continued until the desired thickness is secured.

The Pavin de La Farge pipes are constructed according to one of the first two methods used for Monier pipes.

These pipes are reinforced with longitudinal bars about which are circumferential bars wound in the form of spirals for the smaller size pipes, but for the larger size circular hoops are formed by welding the ends of the hooping bars together. The reinforcements for horseshoe conduits and other sections are made in much the same way. The pipes are usually constructed in sections of from 3 to 6.5 ft. in length. The sections are con-

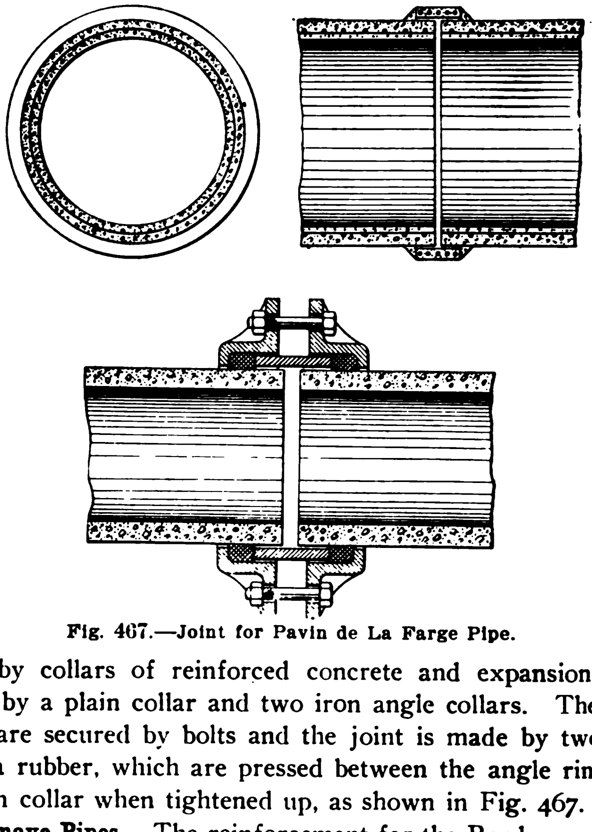


Fig. 467.—Joint for Pavin de La Farge Pipe.

nected by collars of reinforced concrete and expansion joints formed by a plain collar and two iron angle collars. The angle collars are secured by bolts and the joint is made by two rings of India rubber, which are pressed between the angle rings and the plain collar when tightened up, as shown in Fig. 467.

Bordenave Pipes.—The reinforcement for the Bordenave pipe is bent by machinery, and wound in helical rolls, which are placed on a core and adjusted until the required pitch is obtained. The longitudinals are then placed inside or outside the spirals depending upon whether the pressure is from within or without, and tied to them in the proper positions with pieces of wire. Small I-sections and round rods are used for both spiral and longitudinal reinforcements.

The casting is done from a covered platform on which the concrete is mixed. This platform is mounted on a framework which runs on a track over the molding floor, and which carries hoisting apparatus for handling the molds. A core, which is adjustable in diameter and collapsible, together with an outside hollow cylinder divided in halves is used to form the mold. The reinforcing coil described above is set on end, the core placed within it and adjusted to the proper diameter. The outside mold is then clasped in place and the mold is filled with liquid mortar through a special funnel fixed to the platform. Before the mortar is entirely set the funnel is removed and the top of the pipe formed by hand. When the mortar has hardened the shell is removed and the core collapsed and withdrawn. The pipe is left standing for a time and then removed and stacked until used. A quick setting cement is used in the manufacture of these pipes.

Among other notable works in which the Bordenave pipe has been used is that for the water works at Bone, Algeria, built in 1893. Eighteen and a half miles of conduit about 24 ins. in diameter was built, much of it being under a head of from 50 to 80 feet. Figure 468 shows the arrangement of the reinforcement used in this pipe. For a pressure head of 50 feet ft. the coils of the circumferential reinforcements were spaced about 3¼ ins. apart, and for a head of 80 ft. about 2 ins. apart. The longitudinal reinforcements were spaced about 3¼ ins. apart circumferentially. The shell was about 1¾ ins. thick. The reinforcements used were small .47 in. × 2 in. I's, and weighed about 0.142 lbs. per ft.

The joints were spliced with collars of the same construction as the pipe, as shown in Fig. 468. These were slipped over the adjacent ends and fastened by filling the annular space between the collar and pipe with mortar.

Bonna Pipe.—The Bonna pipes are reinforced with a bar having a section the form of a cross. This gives a large area for adhesion. Either a spiral or hooped circular reinforcement is used, depending upon the size of the pipe. Figure 168, page 269, shows a simple spiral wound pipe reinforcement. When hoops are employed they are cut to the proper length, bent into hoops and the ends fastened together with a riveted joint. A series of hoops are placed upright at proper distances apart in frames, and the longitudinal bars having notches cut in their sides at

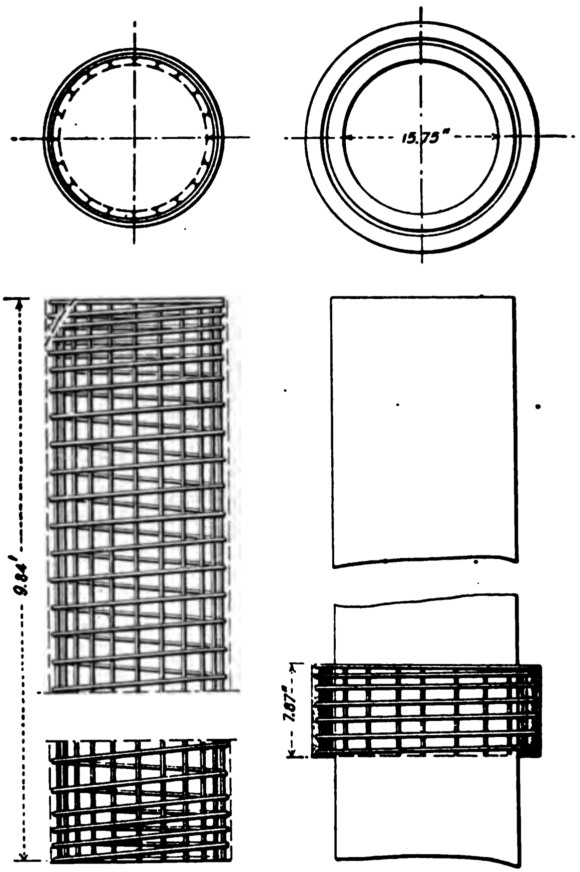


Fig. 468.—Bordenave Construction for Pipe.

proper intervals apart to receive the hoops, are placed in position, and all firmly wired together.

For high pressure, two series of reinforcing skeletons are employed with a sheet steel tube between them. These tubes are also used inside pipes with a single reinforcing skeleton work, in which case they act as a mandrel on which the pipe is molded.

The steel sheet tubes are usually formed of three pieces having their edges bent in the form of clips, as shown in Fig. 469.

These clips are hooked together and clasped tightly by special machinery. The joints are sometimes soldered to make them entirely watertight. For pressures up to 50 ft. the steel tubes are formed of metal of about No. 10 gauge, and for pressure greater than 50 ft. about No. 7 gauge is employed.

The pipes are molded in a manner similar to that used in casting the Bordenave pipe. The cores are of sheet iron and collapsible. The outside mold is made of sheet steel in halves, and held together by hoops. When an inside shell is used it is employed for a core. A liquid mortar formed of a mixture of quick and slow setting cement is poured in much the same manner

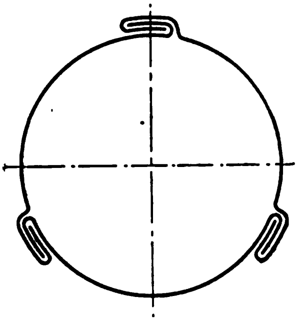


Fig. 469.—Steel Lining for Bonna Pipe.

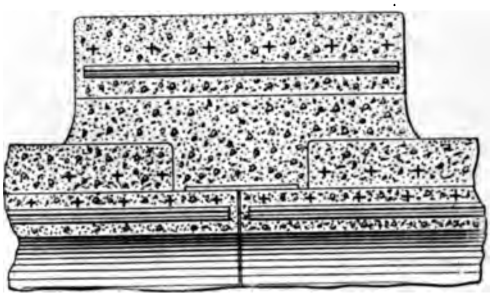


Fig. 470.—Joint for Unlined Bonna Pipe.

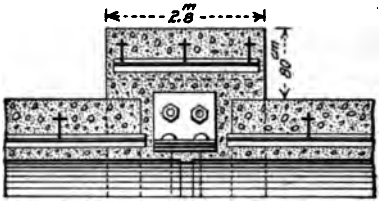
as in the Bordenave system. While the liquid mortar is being poured the sides of the molds are pounded with a wooden mallet to drive out the air and consolidate the mortar. In some cases a steel core with both inside and outside reinforcements is used. In this case the steel shell is put in place, the inside and outside reinforcements put inside and outside the shell; a collapsible core put inside and a sheet steel mold outside, and the pipe poured as before. A traveling platform and apparatus for pouring the concrete are provided as in the Bordenave system.

M. Bonna built a number of large sewers in Paris. In the Mery, Prerielage and Treil districts alone about 75 miles of pipe was laid. These pipes vary from 12 to 43 ins. in diameter, and in some cases 6.5 pipes were used.

The pipes were constructed in lengths of about 8 ft. and laid by setting them end to end and joining them with special collar joints.

Figure 470 shows joint used for joining pipe without interior tube, and Fig. 471 shows detail of joint for pipe with interior tube.

The cost of the conduit having a diameter of about 6 ft. was



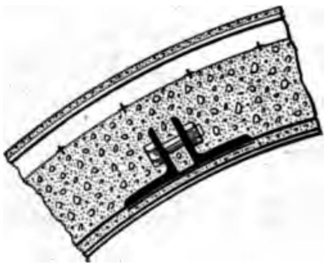


Fig. 471.—Joint for Lined Bonna Pipe.

approximately $18 per lineal ft. for the lined pipe, and about $12 per lineal ft. for the unlined pipe.

U. S. Reclamation Pipe Tests.—Experiments were made in 1904 by the U. S. Reclamation Service upon large reinforced pipes to determine their fitness as service conduits for irrigation work. An attempt was made to determine the various conditions which should be considered in the construction of large pipes. A number of pipes 20 ft. in length and 5 ft. in diameter were built and tested. These pipes were reinforced with 3/4-in. round circumferential rods spaced 3 3/8 ins. centers, and held in place by 8 longitudinal rods 1/2 in. in diameter wired to them.

The following general precautions should be considered in manufacturing and use of reinforced concrete pipe:

(1) Do not allow the sun's rays to touch the concrete when it is being mixed and placed in the forms. If necessary, build a shed over the work to insure this precaution.

(2) If the reinforced concrete pipe cannot be made continuously by machine, do as much of the hand tamping as possible in radial directions. When the tamping must be done at right angles to the radius of the pipe, either in longitudinal or circumferential directions, avoid as far as possible the formation of seams or cleavage planes, from delays in placing the forms and adding fresh material. By making the concrete very wet, delays will not be so dangerous as in the case of dry concrete.

(3) Be careful in tamping not to spring the longitudinal rods, and use as few of these as will suffice to hold the circumferential rods in place, except in case of vertical curves in the pipe, when additional rods or steel cables must be used on the longer sides of the curved part of the pipe.

(4) Do not depend upon the tensile strength of the concrete but make the steel rods of such size and distance apart as will insure no greater stretch of the steel than 0.04 in. in any rod from the maximum pressure to which the pipe is subjected.

(5) Make pipe 1½ in. larger inside diameter than required, to allow of putting two coats of plaster on the inside.

(6) As soon as the pipe is completed give the inside one coat of plaster ½ in. thick, composed of 1 of cement to 1½ of sand, and a small quantity of lime paste, thoroughly cooled, to retard setting. Keep pipe well wet ahead of the plastering. When this coat, which may be left rough, is dry, put on another coat about ¼ in. thick of plaster composed of 1 part of sand to 1 part of cement. This coat should be troweled to a smooth surface, and when it is dry give the entire surface of the pipe a thick wash of fine cement and water.

(7) Provide for drainage of water which may leak through when the pipe is first filled, so that sufficient water may not remain in the trench to soften the ground under the pipe.

(8) Bury the pipe under ground so that there will be no place less than 2 ft. between the top of the pipe and the natural surface of the ground.

(9) In very cold climates provide means for draining the pipe so that it can be emptied at the end of each irrigating season.

(10) A soap and alum mixture may be used to advantage in the making of the concrete, but reliance for impermeability must be placed on the plastering rather than on the material of the pipe.

(11) Do not use reinforced concrete pipes for heads over 70 ft.,

except for short distances, where 100-ft. head might be allowed by taking special precautions.

Jackson Pipe.—The Reinforced Concrete Pipe Co., of Jackson, Mich., manufactures and sells a reinforced concrete pipe (Fig. 472). This pipe was used in the construction of sewers at St. Joseph, Mo. The thickness of the wall of the pipe is 4 ins. for 36-in., 4½ ins. for the 42-in., 5 ins. for the 48-in., and 7 ins. for the 72-in. pipe. Each pipe section is reinforced longitudinally by 5 bars, except the 72-in. pipe, which has 7 longitudinal bars. Two transverse circular bands, each placed 9 ins. from the ends of the section, are also used and the longitudinal bars pushed through slots punched in the hoops. The pipe is manufactured

Fig. 472.—View of Jackson Pipes.

in 3-ft. sections. The thickness of one end of each section is reduced by a rectangular rebate, and by a beveled edge, both extending around the circumference. The other end is correspondingly flanged, so that when two sections are placed end to end they fit together.

The longitudinal reinforcing bars in each section extend with hooked ends into the rebated space which forms the outside groove when two sections are placed together. The sections are then interlocked with a tieband passing completely around the section of the grooves at the joint and through the hooked ends of the longitudinal reinforcing bars. After the sections have been thus interlocked the joint is enclosed, except some 20 ins. on the top, with a galvanized iron shield. The inner surface is surrounded with a galvanized iron mold and the joint poured with 1 : 2 Portland cement mortar.

In the process of manufacture of the pipe a bottom plate of cast iron is used, shaped so as to give the desired flange sections at the bottom end of the pipe. The inside form or core is in four sections of rolled sheet steel. The longitudinal reinforcing rods are inserted in receiving sockets in the plate and the outer case is then added together with lower and upper flange mold. The reinforcing bars are held in place by clips at the top. The circular reinforcing bars are slot punched to receive the longitudinal rods. When the forms are in position the concrete—usually a 1 : 2 : 3 mixture—is placed in small quantities and thoroughly tamped.

Wilson & Baillie Cement Pipe.—The manufacture of cement pipe is not a new industry. As early as 1825 Portland cement pipe was made in England, and since that time it has been used to a limited extent in continental Europe. Cement pipe was first introduced in Brooklyn, N. Y., about 1860, and when proper materials have been used, no failures have been known. Among the advantages of this class of pipe are the convenience of making repairs, the facility with which the pipe may be laid to line and grade, and the ease of making joints. Another advantage lies in the fact that the flat base gives a uniform bearing throughout, rendering the use of a concrete cradle unnecessary, the pipe possessing sufficient inherent strength to withstand any pressure that may be brought to bear upon it.

The early pipes laid in Brooklyn, N. Y., and other cities were all made by hand. The forms in many cases consisted of a solid core and a shallow flask in two parts. The flask was clamped together, placed on a plate or ring and lowered over the core. A tray was fitted on top of the collar and concrete shoveled into the tray and spaded and tamped into place with an iron rammer. The mix consisted of one part Rosendale cement, and three parts of coarse sand, mixed with about 25 per cent. water to the consistency of a thick paste.

Owing to the difficulty of thoroughly mixing and tamping the ingredients, great trouble was experienced in turning out homogeneous pipe of equal density throughout. Up to 1890, all the cement pipe laid in Brooklyn, N. Y., had been made by hand, at which time the Wilson & Baillie Manufacturing Co., who had been experimenting for a number of years, introduced a machine-made pipe. In the manufacture of the machine-made pipe, the cement, sand, broken trap rock and water were measured, and thoroughly mixed in a mill, evenly fed to the moulds and rammed by machinery regulated to produce continuous and uniform blows of any impact desired. The result

Fig. 473.—Machine Head Down Shell in Place.

Machine with Pipe Complete.

is a product perfectly homogeneous and of equal and great density throughout.

Figure 473 shows the machine with head down, shell in place, ready for making a 12-in. round flat base pipe. It also shows the pipe complete, rammed up in the shell. Figure 474 shows the core withdrawn, head raised, shell and pipe removed and the machine in position to receive an empty shell.

A machine similar to the one shown is used for the manufacture of egg shape pipe. Described briefly, it consists in the main of a base with a revolving table and two upright columns, about four feet apart, bolted to the base by flanges and carrying a cross piece on top for the bearings of the gears. There is one large central shaft running to the central head with a key way from end to end, and a key in a cogwheel to set same. This shaft actuates a barrel cam inside of the frame which imparts the tamping or up and down movement to the rammers. The head or frame has a vertical movement in slides secured to the columns, allowing the head to move up and down as the pipe is being formed. By means of a slide with a roller, a small shaft, running parallel to the large central shaft, moves a second cam which has the exact opposite of an egg shape. Attached to cam No. 2 is a lever, the lower end of which is secured to a finger slide. The slide extends into the head which acts as a guide and the finger projects downward and through a shoe

Fig. 474.—Machine Ready for Another Pipe.

carrying the rammers, while the shoe itself is held on a horizontal slide attached to the drum slide which does the tamping.

As the upper cam revolves it causes the upper slide to move in and out, and also imparts, by the lever, the opposite movement to the lower finger slide. The latter propels the shoe carrying the rammers, and thus causes the rammers to conform to the irregular shape of the pipe which is revolving on the table.

For making egg shape pipe there are eight tampers made of the best tooled steel, each running two hundred tamps per minute. As there is only one rammer down at a time, the weight of the head must be borne entirely by the density of the material forming the pipe and this results in an even and regular product such as has been impossible, heretofore, to achieve by hand.

The mixing of the concrete is done mechanically, a cube mixer being used which discharges about 30 inches above the floor, the feeding point being on the floor above. Over the mixer is a hopper into which the crushed stone and sand are discharged by bucket conveyor from the ground. The cement is distributed into the hopper, through the mass. The whole is discharged into the mixer dry, where it receives several revolutions before the water is added, which is allowed to enter in the usual way through the hollow shaft of the mixer. The proportions used in the mix are one and one-half parts Portland cement, one part sand, and three parts trap rock screenings, containing 20 per

Fig. 475.—Showing Manner of Stripping Shell from Pipe.

cent. of stone dust. The amount of water used in the mass will vary from 10 per cent. to 15 per cent., according to the dryness of the ballast. Water is measured in a marked tank and is in control of the man who opens and closes the mixer. The mix is known as a "dry mix," but will ball in the hand with some pressure. The mixed concrete is delivered to the machines in barrows and is fed into the hopper by two men, one on either side. As soon as the flask is full and the core automatically lifted clear, the flask is taken up by a pipe truck having arms with sockets to engage the lugs on either side of the flask and wheeled into the stripping rooms where it is allowed to stand about thirty minutes before being stripped.

Figure 475 shows the manner in which a shell is stripped from the 12-in. round flat base pipe, and also a similar view for a 24-in. egg shape flat base pipe. After the pipes have set overnight a spray of water is turned on and the pipes kept damp until the expiration of six days, when they are removed from under cover and placed in a yard. At the end of 30 days they are crystallized sufficiently to be handled in the work.

Spurs for house connections are made as follows: A hole is cut at the proper point on the side of the pipe and a center is placed in the interior; cement mortar is then spread over the form and the connection piece is bedded in place and a heavy band of mortar is wiped around the joint on the outside. After the center is removed, the inside joint is finished with a trowel.

Fig. 476.—Cement Pipe with Connection for Lateral.

Figure 476 shows 15-in. egg shaped pipe with manner of making house connections.

In order to meet hydrostatic tests, the interior of the pipes are coated with California maltha reduced in bisulphide of carbon. Satisfactory results in this direction have also been obtained by using silicate of soda as a bath into which the pipes are plunged.

These pipes are all 3 ft. in length, with hub joints, with the exception of the 6-in. size, which is 2 ft. 3 ins. long. The thickness of walls is as follows:

6-inch round pipe	¾ inch.
9-inch round pipe	⅞ "
12-inch round flat base pipe	1 1/16 "
15-inch egg shape flat base	1⅜ "
18-inch egg shape flat base	1⅝ "
24-inch egg shape flat base	2 "

Collars are of the following dimensions:

6-inch	collars	1 5/16	in.	deep	with	joint	of	1/8 inch.
9 "	"	1 5/16	"	"	"	"	"	1/8 "
12 "	"	1 3/8	"	"	"	"	"	3/16 "
15 "	"	1 5/8	"	"	"	"	"	1/8 "
18 "	"	1 5/8	"	"	"	"	"	3/16 "
24 "	"	1 3/4	"	"	"	"	"	1/4 "

Figure 477 shows a length of 24-in. egg shape flat base pipe. Some interesting tests have been made on the crushing strength

Fig. 477.—Egg-Shaped Cement Pipe.

of machine-made cement pipe during the past year, a summary of which follows:

Size and Description.	Breaking Weight.
12-inch round and flat base	10,624 lbs.
18-inch egg " " "	*18,785 "
18- " " " " "	12,287 "
18- " " " " "	†13,190 "
24- " " " " "	26,547 "

*Cracked at 10,155 lbs.; additional required to crush.
†Cracked at 9,717 lbs., additional required to crush.

In making the tests, a wooden beam, 20 ft. long, was used, with a 2-ft. fulcrum. The pressure was applied to a saddle having a rubber gasket between it and the pipe so as to give the saddle an even bearing and thus do away with any concentrated pressure.

Some vitrified pipe tests at the same time showed the following results:

Size and Description.	Breaking Weight.
12-inch double strength shale	7,756 lbs.
12-inch single strength vitrified	7,544 "
12-inch standard Akron (average of 3)	5,500 "
12-inch vitrified	7,859 "
18-inch Akron double strength	8,842 "

It is probable that the old aversion to cement pipe will be gradually dissipated by the introduction of an improved machine-made product, and that the use of such pipe will be largely increased during the next few years.

Reinforced Concrete Conduits Built in Place.—Great care must be taken in the construction of sewers and aqueducts in order that they be strong and as nearly impervious as possible. The centers must be smooth, strong and rigid, and so constructed that they are easily assembled and taken down.

Examples will be given to illustrate the most successful applications of reinforced concrete to various forms of concrete sewers and aqueduct construction. The method of construction and a description of the forms used are given in connection with the description of various structures.

Simplon Aqueduct.—The aqueduct which carries water from the Rhone to the power works of the Simplon tunnel at Breig, Switzerland, is of Hennebique construction. It is rectangular in sections and 1.86 miles in length, with a fall of 6.35 ft. per mile. It is carried on supports spaced about 16 ft. 5 in. centers.

Figure 478 shows transverse and longitudinal sections of half a bay, giving details, general dimensions, and materials used. Sizes and dimensions given in this figure are in the metric system.

The inside dimensions are about 6 ft. 3 ins. by 6 ft. 3 ins., and the side walls are about 4 ins. thick. The bottom has a thickness of about 4 ins. at the sides and is thickened toward the center to nearly 6 ins. The roof is thickened in the same manner from 3⅛ to 4¾ ins. The vertical reinforcing rods in the walls are about ⅜-in. diameter, and are spaced 7⅞-in. centers. Three longitudinal rods were used at the top, bottom and sides, the top and bottom rods being about ⅝-in. diameter, and the side rods $\frac{5}{16}$ in. in diameter. A ⅝-in. trussing rod which at the center of the bay is at the bottom of the sides, is bent upward, runs to the top at the supports.

The bottom reinforcement consists of two sets of rods about $\frac{3}{8}$ and $\frac{5}{16}$ in. in diameter, both sets being spaced $7\frac{7}{8}$ in. apart. The $\frac{3}{8}$-in. rods are bent up and form the side reinforcement, and the $\frac{5}{16}$-in. rods are also bent up and extend 1 ft. 6 in. into the sides. There are two longitudinal rods 5-16 in. in diameter spaced evenly on each side of the center. The top is reinforced by $\frac{1}{4}$ and $\frac{5}{16}$-in. rods spaced $7\frac{7}{8}$-in. centers, and located as shown in the figure. Stirrups of $\frac{3}{4} \times \frac{1}{32}$-in. hoop iron are placed about all rods.

The roof was designed to carry a load of 165 lbs. per sq. ft. and an internal pressure of 62 lbs. per sq. ft. Provision for

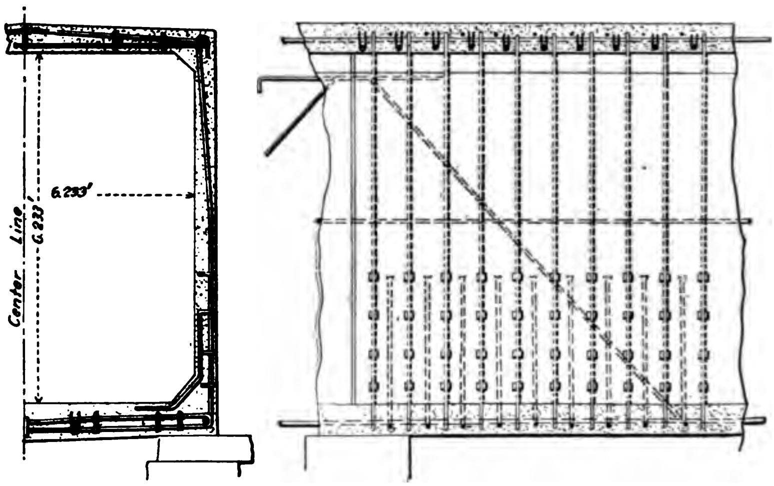


Fig. 478.—Aqueduct for Simplon Tunnel Water Power Plant.

expansion was made by open joints over the piers. These were filled after the concrete had set and before the water was turned into the flume. When filled with water very little expansion or contraction can occur. The cost of this aqueduct was about $5.90 per linear foot.

Salt Lake City Aqueduct.—A reinforced concrete aqueduct 38,000 ft. long has been constructed at Salt Lake City in order to secure an added water supply. It is partly in excavation, partly in tunnel and partly above ground, thus giving three types of construction.

The aqueduct is built on a hydraulic grade and will not be subjected to internal pressure other than the weight of the water flowing in it.

Figure 479 shows normal section in excavation, and consists of a roof slab modified to suit depth of fill. The thickness is

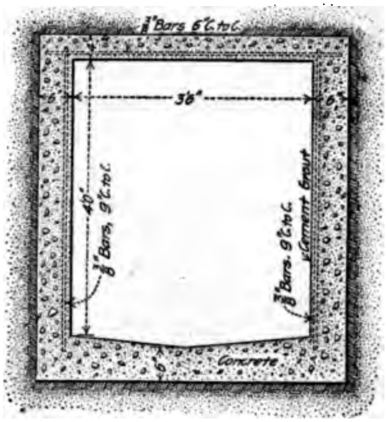


Fig. 479.—Section of Salt Lake City Conduit in Excavation.

4 ins. for 5 ft. fill with Ransome twisted bars as shown; 5 ins. for 5 ft. to 10 ft. fill, reinforced with ½-in. bars spaced 9-in. centers. For a covering exceeding 10 ft. a 6-in. slab reinforced with ½-in. bars spaced 6 ins. apart is used.

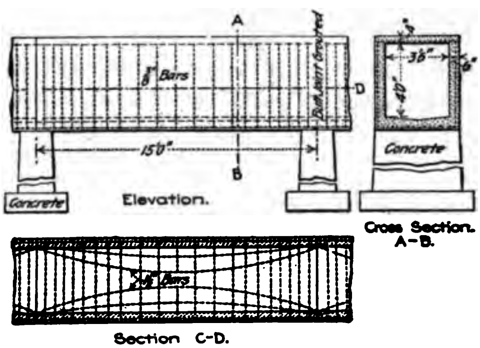


Fig. 480.—Section of Salt Lake City Conduit on Piers.

The tunnel section is in rock and consists of a concrete lining for sides and bottom and brick arch for top. It was deemed unwise to build on an embankment, and section shown in Fig.

480 on concrete piers, spaced 15 ft. centers with additional reinforcement was used. But joints were used at each pier, and joints grouted in cold weather so as to throw the conduit into compression to care for expansion. The interior of the conduit was coated with a grout of a 1:1 Portland cement and fine sand.

The concrete used was a 1 : 2½ : 4 mixture of Portland cement, sand and broken stone or gravel. The engineer in charge was Mr. Geo. W. Riter, City Engr. of Salt Lake City.

Cedar Grove Reservoir Conduit, Newark, N. J.—A 5 ft. reinforced concrete conduit 4,000 ft. in length was employed in the construction of the Cedar Grove reservoir for the Newark, N. J., water works, and extended from the regulating inlet gate chamber at the north end of the reservoir to the inlet stand pipe at the south end.

Another line consisting of a double conduit 1,500 ft. long

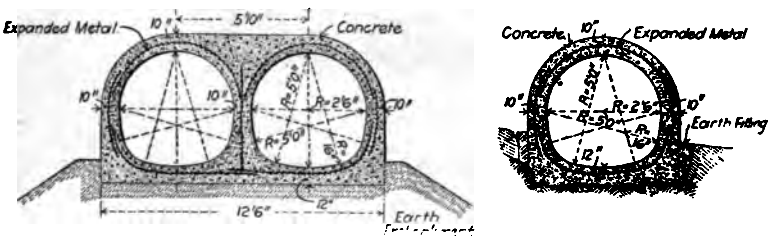


Fig. 481.—Conduits, Cedar Grove Reservoir, Newark, N. J.

extended from the outlet gate chamber to the outlet channel. Figs. 481 and 481A show cross sections of these two conduits. The reinforcement consisted of a circumferential ring of No. 10 gauge 3-in. mesh expanded metal with lapped joints.

A 1 : 2 : 5 Portland cement concrete with 1½-in. broken stone was used.

The center used in the construction of the single conduit is shown in Fig. 481A. These centers were built in sections 16 ft. long, and were supported on brick piers resting upon foundation concrete as shown by the drawing. The forms were set up and the lagging put in complete, except for a manhole at the middle of the crown. Through these holes the concrete for the inverts was passed to the men working on the inside of the form. After the invert concrete was put in the manholes were closed and the concreting completed in the usual manner. To collapse the forms the bolts holding the bottom cross pieces were withdrawn, the cross pieces removed and the feet of the ribs slightly drawn together. A certain amount of play was allowed in the joints to make them slightly flexible. The centers were struck in about 36 hours after the concrete was laid. The method of building the double conduit is shown in Fig. 482.

Reinforced Concrete Conduit for the Jersey City Water Supply Co.—The conduit for the Jersey City Water Supply Co. is an excellent example of reinforced concrete construction. Some three and a half miles of reinforced conduit 8½ ft. in diameter was used. Four principal types of reinforced sections were employed. Fig. 483 shows sections used in rock, in stiff earth and

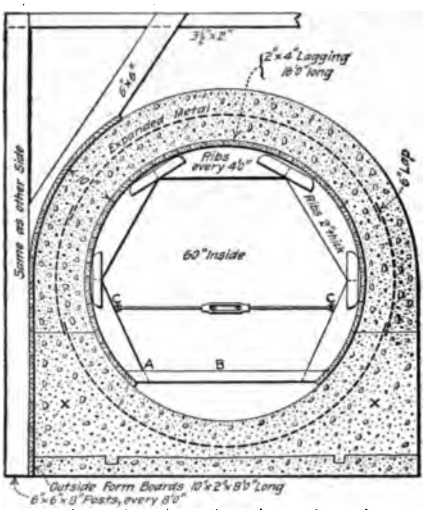


Fig. 481A.—Forms for Single Conduit, Cedar Grove Reservoir.

rock, in soft earth bottom and on foundation embankment. These various sections were reinforced with transverse ⅜-in. square steel rods bent in the form shown in the drawing, spaced 1 ft. apart, and ¼-in. longitudinal twisted rods placed 2 ft. apart and wired to the transverse rods. The general dimensions and thickness of concrete used is shown in the drawings. About 90% of the conduit was built in open trench of the type for stiff earth and rock. This section was used for all depths of cover up to about 10 ft. In a few pieces amounting to 826 lin. ft., when the cover was about 15 ft. the heavy rock section was used.

Fig. 482.—Forms for Double Conduit, Cedar Grove Reservoir.

In the construction of both these types the transverse rods were made of such lengths as to extend 1 ft. below the bottom of the outside forms, below which the concrete was built against the hard earth or rock sides of the trench.

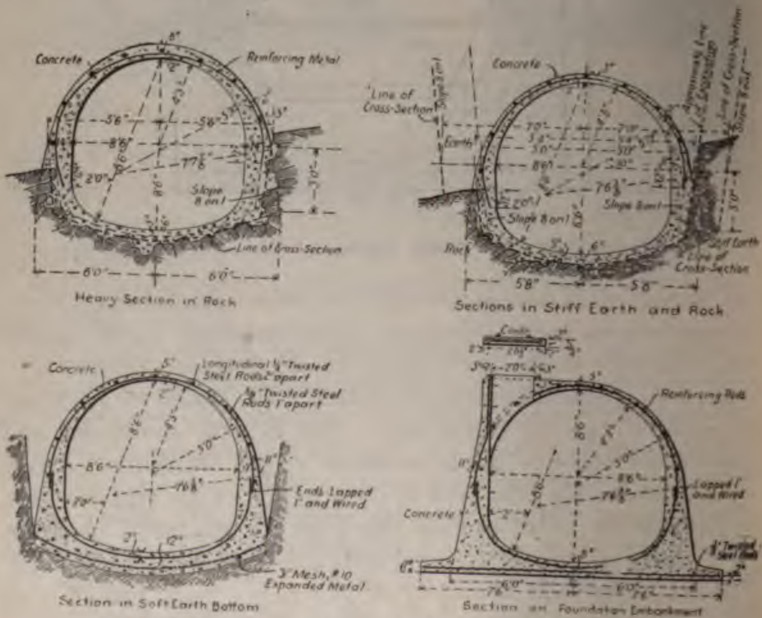


Fig. 483.—Sections of Jersey City Water Works Conduit.

The soft ground section shown was used for short lengths where the bottom of the trench was in soft earth. Concrete 6 ins. in thickness at the middle and from 8 to 18 ins. thick at the sides, according to the conditions, in which 3-in. mesh No. 10 expanded metal was imbedded, was placed as a foundation and the conduit built upon it as shown in the sections. Some 420 ft. of this type of section was employed, the ⅜-in. transverse reinforcing rods in these last two types completely encircling the base of the conduit. In all cases where the rods were spliced a lap of 1 ft. was made and the ends wired together.

Fig. 484.—Center for Jersey City Water Works Conduit.

A very dense concrete weighing from 156 to 160 lbs per cu. ft. was used. It was composed of 1 part Atlas Portland cement, and 7 parts of aggregate consisting of a carefully balanced mixture of sand and crushed trap rock, crusher run of 2 ins. maximum size stone. A very wet mixture was used, it being of such a consistency that it would just flow through chutes with a slope of 1 vertical to 8 horizontal.

The concrete was run through chutes from the mixer to watertight dished boxes, from which it was shoveled with coal scoops into the forms. Two types of centers, as shown in Figs. 484 and 485, were employed. That shown in Fig. 484 was covered with thin sheet steel. All centers were made of eleven (11) segments each weighing about 200 lbs. The specifications required that the conduit should be built in monolithic sections not exceeding 20 ft. in length, but sections 50 ft. long were used where the top of the conduit was not over 18 ins. above the undisturbed ground. Where the trench was near the required width no back forms were used. When it was necessary to use them they were built up in sections 2 ft. wide and 12.5 feet long. No outside forms were employed for a width of about 5 ft. at the top of the arch. The forms were built up of ⅞-in. lagging with ribs and bracing as shown in the illustration.

Fig. 185.—Center for Jersey City Water Works Conduit.

The general method of construction was about as follows: Forms were set up for the sections to be built, making a continuous inner mold for the conduit, except for about 6 ft. in the lowest part of the invert. These forms were supported on 6 in blocks of concrete which were built into the work. Each 12.5 ft. section of the inside forms had a scuttle about 2 ft. square at the crown of the arch. After having been set the forms were greased with cheap vaseline cut with kerosene, and the twisted rods put in position and wired together. The concrete was deposited on the outside, and by means of tamping bars forced under the forms until it appeared on the inside and filled the entire space between the ground and the forms. Concrete was then thrown through the scuttles and screeded into shape and finished smooth by troweling while the mortar was plastic. The scuttles were then closed and the work of making the concrete shell continued. Outside forms were added as the work progressed until all the concrete had been placed, except about 5 ft. along the crown; this was completed without the aid of an outside form. Care was taken to keep the forms clean. At the end of

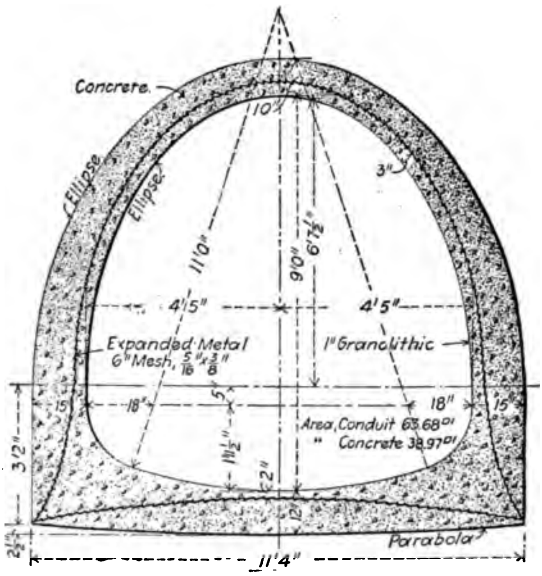


Fig. 486.—Cross-Section of 9-Ft. Concrete Sewer, Torresdale Filler Conduits.

each day's work a groove was formed in the face of the concrete around its whole ring to bond with the concrete next laid. The reinforcing rods extend across these joints. Fine cracks were found to girdle many of these joints, but were observed nowhere else. The leakage, however, was not serious, and after a time the cracks silted up. Little or no brushing or plastering of the concrete surface was done and the interior of the conduit was found to be smoother than average brick work in similar construction. The speed of construction was quite rapid. Between July 25 and November 14, 1903, 18,500 ft. of conduit was built.

On one section a force of 38 men averaged 39.8 ft. of conduit per day for 65 days.

The Torresdale Filter Conduits.—Large reinforced concrete conduits and sewers were used in the construction of the Torresdale filters for the Philadelphia water supply. The effluent conduit, 2,200 ft. long, has the form of cross sections shown in Fig. 486 for the 9 ft. sections. It is in successive portions 7½, 9 and 10 ft. in height. The discharge conduit 850 ft. in length is also ten (10) ft. in height. These conduits will be under a head of 20 ft. Two sizes of concrete sewers were used: first, a 6-ft. reinforced section 1,800 ft. in length and an 8½-ft. sewer 300 ft. in length reinforced, as shown in Fig. 487, with two layers of 3-in. mesh No. 10 expanded metal; one was placed

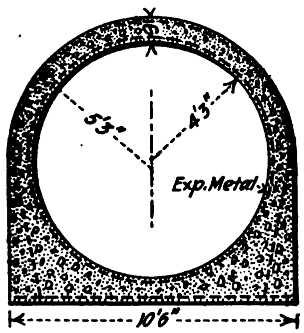


Fig. 487.—Cross-Section of 8½-Ft. Sewer, Torresdale Filler Conduits.

2½ ins. from the interior surface and the other 2½ ins. from the outside.

The reinforcement for the discharge conduit consisted of one layer of 6-in. mesh No. 4 expanded metal cut double width, but as it proved difficult to cut such heavy metal, two layers of No. 4 metal 6 in. mesh wired together was used for the filtered water conduit. The position of the metal is shown in the drawings. Adjacent sheets were lapped with not less than 6 ins. The 6-in. metal proved much more satisfactory than the 3-in., as the latter had a tendency to screen the mortar from the stone. Some difficulty was experienced in properly placing and ramming the concrete in the bottom and lower portion of the circular sections, much better work resulting when the "horseshoe" section with its comparatively flat bottom was used, as the metal could be kept closer to shape and the concrete more thoroughly rammed.

The general character of the forms and centers used for both the conduit and sewers are clearly shown in Figs. 488 to 492.

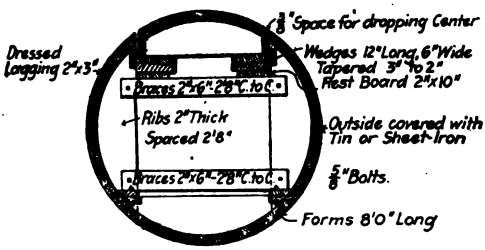


Fig. 488.—Form for 6-Ft. Sewer.

The 9-ft form was the last built and proved the best. The forms were covered with No. 27 galvanized sheet iron, which

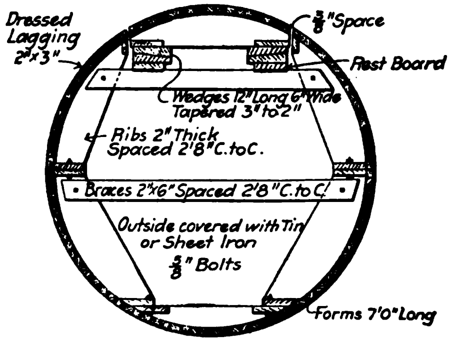


Fig. 489.—Form for 8½-Ft. Sewer.

gave a smooth surface to the concrete and greatly lengthened the life of the forms. Their surface was cleaned and oiled each

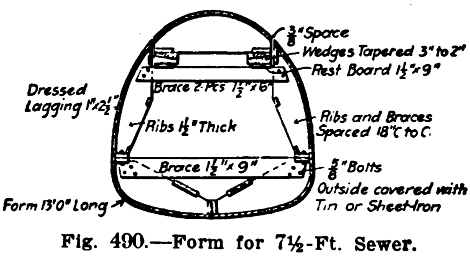


Fig. 490.—Form for 7½-Ft. Sewer.

time before being used. The bracing inside the forms was arranged to allow a center to be taken apart after the concrete had been placed for at least 60 hours, and brought forward through the form already in place for further use. The straight portions of the conduit were built in monoliths 12 ft. to 13 ft. 6 ins. in length with a bonding groove in the end of each section, and with the expanded metal extending from one section to the next.

All conduits have an inch granolithic finish on the interior surfaces composed of 1 part Portland cement, 1 part sand, and 1 part grit, mixed to the consistency of stiff grout and poured just in advance of the placing of each layer of concrete with an inch space between the inside forms and sheets of iron, with lugs which were gradually withdrawn as the concrete was placed. Very smooth surfaces were thus secured.

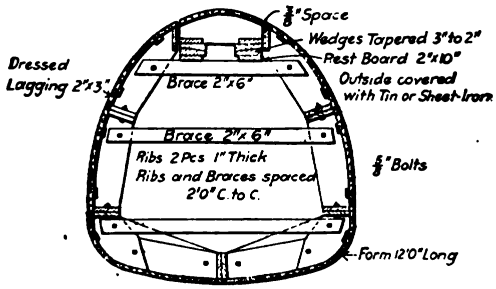


Fig. 491.—Form for 10-Ft. Sewer.

The concrete was mixed rather wet and placed in 6-in. layers. The manner of building a section was as follows:

A bulkhead (Fig. 493) was set up the length of the form in advance of the end of the section already built, and the bottom concrete filled in to within 1 in. of the invert face. The bottom section of the form, which is in two pieces, is then set, its rear end being bolted to the face of the last form used and its front end resting in the bulkhead.

About 2 tons of pig iron were then placed on the invert form to keep it from floating, and the granolithic mixture was then poured through four spouts at the corners into the space between the concrete and the form. The centers were then put in place and the face finish and concrete put in as already explained. The bulkhead had a slot permitting the expanded metal to project 6 ins. from the face of the concrete to tie the adjacent sections together, and also a rib to form a depression into which the concrete of the next section was rammed.

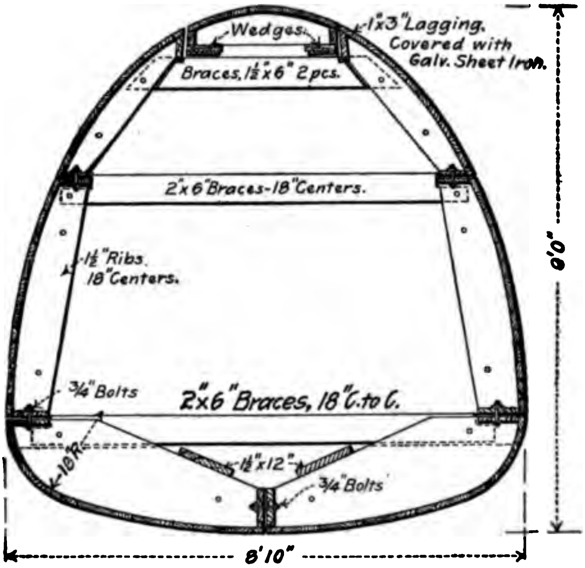


Fig. 492.—Form for 9-Ft. Conduit.

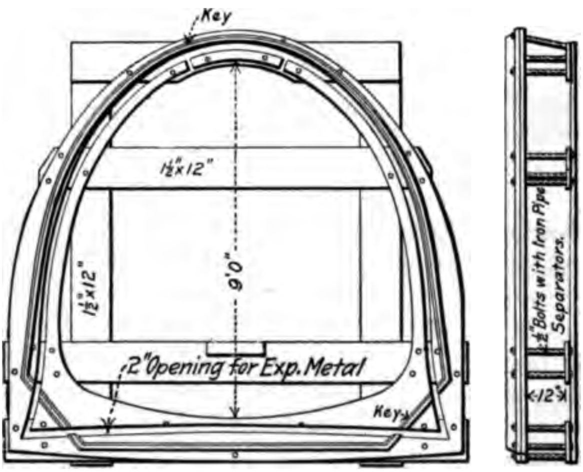


Fig. 493.—Bulkhead Form, Torresdale Conduits.

It was found that the different sized sections could be built in about the same time, it taking usually from 8 to 10 hours to build a section. One foreman and 18 men on top of the trench mixed and handled the concrete and granolithic mortar, while 1 foreman, 1 carpenter, and 7 men in the trench set the forms, placed and rammed the concrete, etc.

It required 20 cu. yds. of concrete and 1,200 sq. ft. of expanded metal and 125 bags of cement to build a section 13½ ft. long. The cost of the conduit, excluding the cost of excavation and the contractor's profit, but including forms, expanded metal, materials for concrete and labor, is stated as about $10.50 per cu. yd. The cost per linear ft., exclusive of excavation and

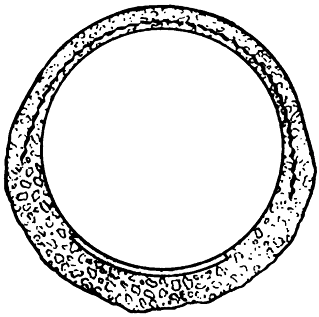


Fig. 494.—Sewer at Providence, R. I.

engineering supervision of the different sizes of conduit and sewers, was as follows:

Size.	Cost per lineal ft.
6 feet	$7.24
7½ feet high	19.85
8 feet diameter	22.52
8½ feet	21.59
9 feet high	23.94
10 feet "	26.37

The Providence Sewer.—Figure 494 shows a section of a reinforced concrete sewer in Brooks Street, Providence, R. I.; 520 ft. of it is 56 in. in diameter, 830 ft. is 48 in. in diameter, and 2,150 ft. is 36 in. in diameter. The reinforcement consists of expanded metal No. 14 gauge, 4 in. mesh. A piece 18 ins. wide is placed in the concrete at the sides of the sewer with about half its width below the springing line. The crown reinforcement laps 6 ins. on these side pieces.

The minimum thickness used for the concrete was 4 ins. The invert for about one-fourth the circumference of the sewer was lined with granolithic work similar to that used on sidewalks. Sheet steel was used for inside forms for both upper and lower halves of the sewer. Sections 8 ft. long were lubricated with crude oil before setting them in place. The forms were left in place 24 hours, and 32 ft. of sewer was made a day with a gang of 10 men, the bottom half being kept 1 day ahead of the arch.

The Harrisburg Intercepting Sewer.—The Harrisburg sewer is a good example of a reinforced concrete sewer with a horseshoe section. A description of the method of construction used will serve as a good illustration of sewer construction in which the invert is laid by a template. A typical section of this sewer is shown in Fig. 495.

The invert is the arc of a circle with a tangent at an inclination of 3 to 1 on each side. The arch is a parabola. The upper

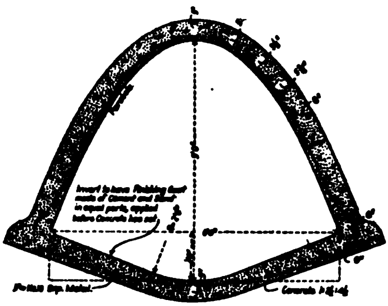


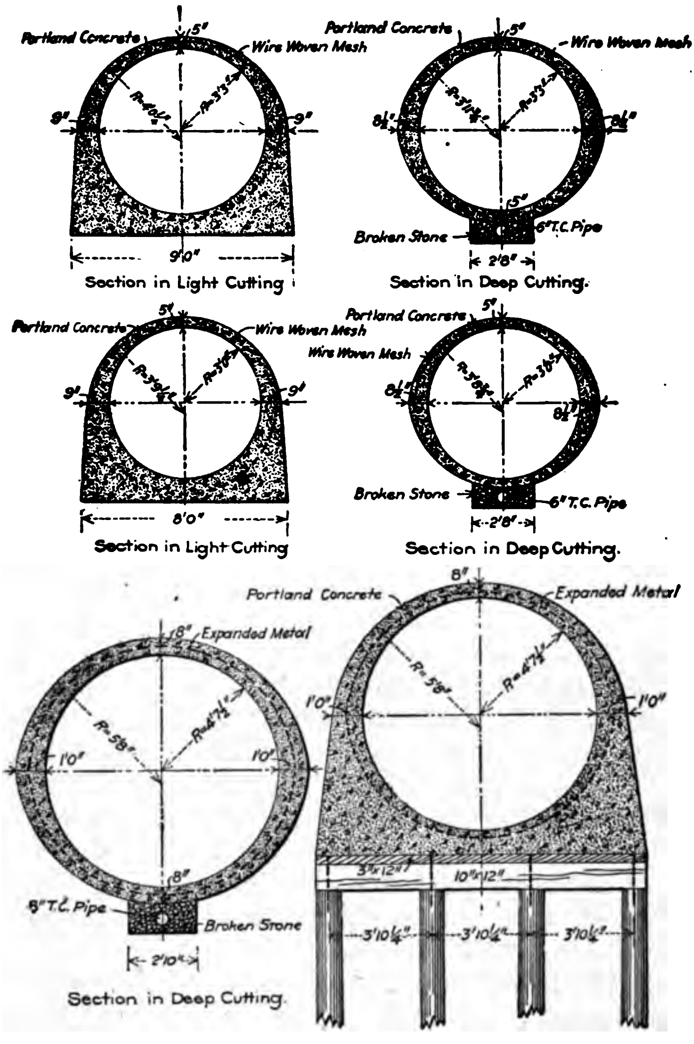
Fig. 495.—Harrisburg Intercepting Sewer.

7,600 ft. of this sewer is 3 ft. 9 ins. high by 5 ft. 1½ ins. wide at the base of the arch, and has an area of 12.278 ft. The lower portion is 5 ft. high by 6 ft. wide, inclosing an area of 16.335 sq. ft. The general dimensions and thickness of the concrete are shown in the drawing for the 6-ft. section. The same thickness of concrete and same reinforcement are used for the 5 ft. 1½ in. section, the general dimensions alone being changed. No. 10 gauge 3-in. mesh expanded metal, located as shown, was used for the reinforcement. The metal, as will be seen, is not placed so that it will provide the greatest tensile strength, but it was considered that the section would have ample strength.

After the ditch was excavated to sub-grade the bottom was shaped to the proper profile and section. A small trench was then dug in the center, below sub-grade, and the underdrain laid with its top about 3 ins. below the invert concrete. A wooden template conforming to the shape of the invert was then accurately set to grade and line, about 12 ft. beyond the end of the completed invert, and the intervening section laid in the following manner: The concrete below the line at which the expanded metal was to be placed was thrown in and tamped. The metal previously bent to the proper shape, was then placed with its ends extending up at both sides for lap with the arch metal, and the top course of concrete put in place. Before ramming it was roughly shaped by means of a 14-ft. straight edge. one end resting on the finished invert and the other on the template. After careful ramming the concrete was covered with a ½-in. coat of 1:1 cement mortar trued by the straightedge and troweled smooth. Each 12-ft. section as it was completed was tested for grade by the inspector. The arch centers were 2½ x 2½ x ¼ in. steel angles bent to the proper shape and spaced 3 ft. 4 in. apart, the ends resting on wooden wedges placed on the side slopes of the invert. Two-inch planed pine lagging 10 ft. long was laid loose on these centers and coated with soft soap. The arch metal previously bent to shape was placed over this lagging and held at proper distances from it by blocking it out with small stones.

A 1:2½:4½ hand mixed Portland cement (Giant Portland brand) concrete was used. A Ransome mixer was tried, but owing to the small quantities of concrete needed for the sewer sections, it was found more economical to do the mixing by hand. From the mixing board the concrete was passed through a chute into a box supported on the lower bracing, from which it was shoveled into place. Wet concrete was used and forced through the metal against the arch center. The outside arch lagging was built up of rough lumber against outside pine ribs. After three days the wedges were removed from under the steel ribs and the centering collapsed. The inside and outside was then gone over carefully and all imperfections in the concrete filled with 1:1 mortar. The back fill was kept 48 hours behind the arch construction. It is stated that the labor cost was approximately 53 per cent. of the total cost of the sewer.

This sewer stood without injury a severe test accidentally brought upon it at a time when the concrete was less than two weeks old. A loaded coal train was derailed on a siding crossing the sewer at an angle of about 20° at a time when the siding was supposed to be temporarily abandoned. The ties were buried out of sight in the soft clay filling by the weight of the train. The cars were left in position for several weeks, but no evidence of failure could be found in the sewer underneath.



Section through Marsh.

Fig. 496.—Price's Run Sewer, Wilmington, Delaware.

Wilmington Sewer.—The Price's Run sewer at Wilmington, Del., is a notable example of reinforced concrete sewer construction on account of the light sections used. Circular sections 9 ft. 3 ins., 6 ft. 6 ins., 5 ft. and 4 ft. 9 ins. were used. Two kinds of reinforcement, expanded metal and wire-woven fabric were employed. Fig. 496 gives cross sections and general dimensions of a 9 ft. 3 in., 6 ft. 6 in. and 6 ft. sewer, two types being used.

The section in "light cutting" was built above the natural surface of the ground, while sections in "deep cutting" were built in deep trenches. The "section through marsh," as shown in the figure, was supported on piles from 8 to 36 ft. long, four piles to the bent, with bents spaced 4 ft. centers. Caps of 10 × 12-in. yellow pine rested upon the piles and supported a flooring made of 3 × 12-in. hemlock plank laid with broken joints.

The reinforcement for the 9 ft. 3 in. sewer was 6-in. mesh No. 6 gauge expanded metal cut in 8 × 5½ ft. sheets and placed in a single layer about the sewer. It was located 2 ins. from the inner face and held carefully in position while the concrete was rammed in place. The sheets were lapped one mesh at both ends and sides. It was only found necessary to wire fast the top sheet.

Wire-woven fabric was used for part of the work. This material was furnished in continuous rolls about 100 ft. long and 5½ ft. in width. The wire was No. 8 gauge with a No. 6 wire for selvage, and the mesh was 6 × 4 ins. The fabric was cut in lengths to entirely surround the sewer and embedded in the concrete as the latter was rammed in place, as was done with the expanded metal, except over the centers, when the fabric, being more pliable than the expanded metal, was held the proper distance from the wooden center by means of 2-in. blocks, which were removed as the concrete was placed. The wire fabric being in one length was more easily placed than the expanded metal, but the latter, being stiffer, was more easily maintained in proper position. The two kinds of metal were used for the purpose of determining the relative values of each. The expanded metal cost delivered 4 cents per sq. ft., and the wire-woven fabric 2½ cents per sq. ft.

The molds were built of 4-in. lagging as the concrete was put in, and the concrete rammed in 4-in. layers. The bottom of the invert was smoothed up with a coat of plaster.

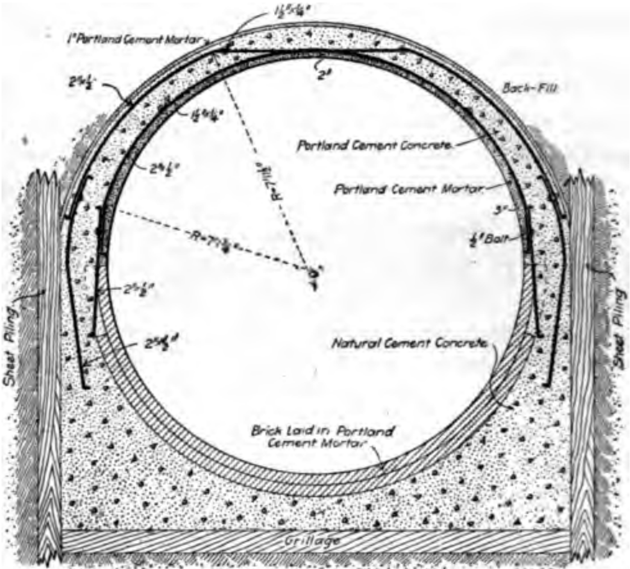
The concrete was mixed moderately dry, and for the invert was composed of 1 part Portland cement, 2 parts stone dust and 6 parts 1½-in. stone. For the arch a ¾-in. stone was used with a 1:2:5 mixture. As smooth work and as good results were obtained when 1½-in. stone was used as when ¾-in. was employed. The stone was a blue granite well selected as to size.

The section used for this sewer, while only 8 ins. in thickness for the crown of the 9 ft. 3 in. sewer, stood without injury the dumping upon it of a cubic yard of earth and rock from heights of from 3 to 10 ft., together with a weight of 25 ft. of loose earth fill.

The section through the marsh with a crown thickness of 9 ins., which is above the natural surface and without any earth covering, was upon two occasions subjected to severe pounding of tons of ice resulting from the breaking up of the ice on the Brandywine River, without any apparent injury. This would seem to indicate that this sewer section was of ample strength.

Cleveland, O., Sewers.—Probably more reinforced concrete sewers have been built in Cleveland, Ohio, than in any other American city. The first sewer built of reinforced concrete is known as the main intercepting sewer, and extends along the lake front. The first section built was 3½ miles in length and is 13½ feet in diameter. The reinforcement consists of longitudinal and transverse steel rods arranged according to what is known as the Parmally system. One section about 2 miles long, is from 35 to 44 feet deep and only 17 feet in the clear from the center line of the Lake Shore & Michigan Southern Railway tracks. This portion was built in open trench. Considerable difficulty was experienced in the construction of this section, as water and quicksand were encountered. Nine-inch sheet piling 28 ft. long was first driven with steam hammers on both sides of the trench, the excavation made by cable machinery and the piles braced by 10 x 10-in. and 8 x 8-in. wale and strut timbers put in position; ordinary sub-strutting and bracing was used in the bottom of the trench.

Figure 497 shows section of the main intercepting sewer. The invert was built of natural cement concrete, and the two anchor rods of 2½ x ½-in soft steel shown at the sides and spaced 15 ins. centers in staggering rows bonded the invert to the crown and strengthened the sides against lateral pressure. The invert was lined with 2 courses of shale brick. The arch centering was placed in the usual manner and the lagging was covered with building paper waterproofed with paraffine. This paper did not prove entirely satisfactory, as it became soaked and swelled, giving more or less of a rough surface. After the centering was placed $2\frac{1}{2} \times \frac{1}{2}$-in. curved transverse bars were bolted to the anchor bars to make an inner and outer skeleton, the first adjacent and parallel to the intrados, and the second flattened on top at the level of the crown. To these bars were bolted



Section of Main Intercepting Sewer

Fig. 497.—Main Intercepting Sewer, Cleveland, O.

8 lines of horizontal longitudinal $1\frac{1}{2} \times \frac{1}{4}$-in. bars. Portland cement mortar 3 ins. thick was then laid on the lagging enclosing the inner row of bars and formed a finished surface for the arch soffit, through which none of the concrete stone could penetrate. Before this mortar set concrete was rammed in between it and the sheeting to a height of 18 ins. above the springing line, and the remainder of the concrete rammed in place against the 3 ins. of mortar without the use of outside forms. The upper surface of the concrete was finished with 1 in. of Portland cement mortar. The arch concrete was made of 1:3:7½ mixture with

$1\frac{1}{2}$ in. screened broken stone. When the voids in the stone exceeded 40 per cent. the proportion was 1 : 3 : 6. Back filling was commenced as soon as the concrete was from 6 to 12 hours old, but the centers were not removed for two weeks.

The section shown in Fig. 498 was used for the Gilbert street sewer. Round bars instead of flat ones were used, the connections being made by hooking the bars together instead of bolting them. As will be seen, the arrangement of the bars is somewhat different from that shown in Fig. 497, in which flat bars were used. The primary bars are nearly horizontal at the crown where they

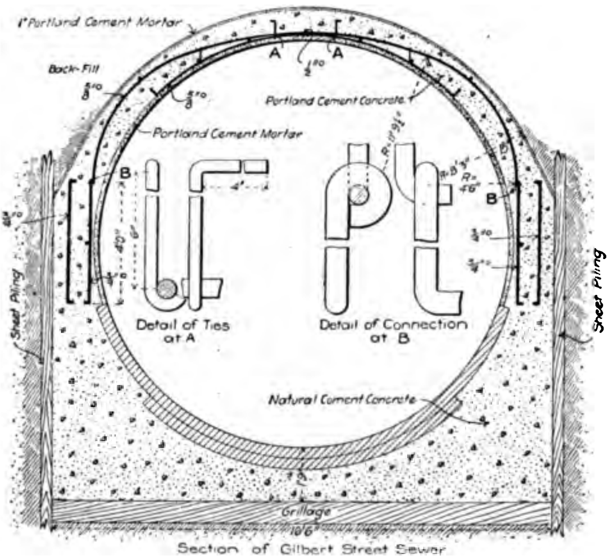
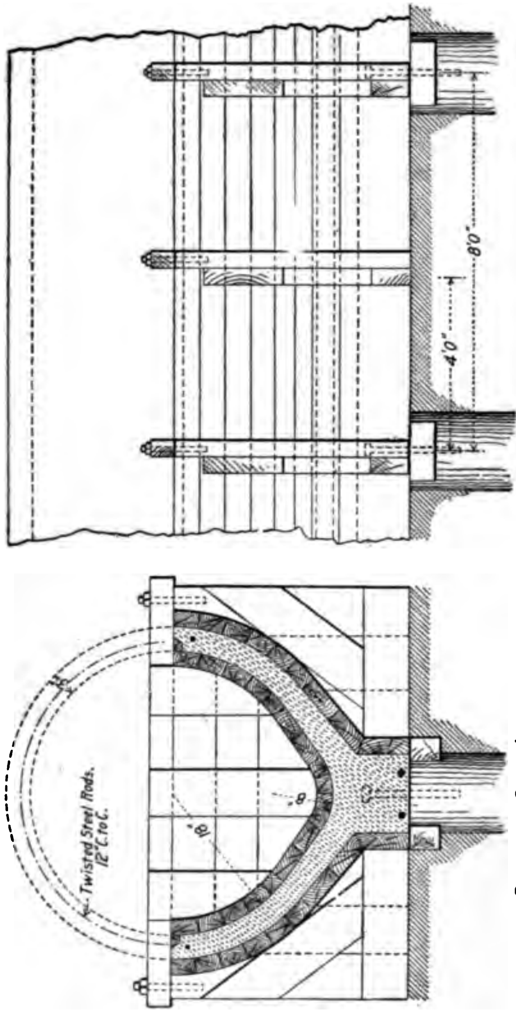


Fig. 498.—Gilbert Street Sewer, Cleveland, O.

pass near the intrados and thence extend through the arch to the extrados along the haunches and then back again through the arch to the intrados, where they are anchored to the vertical side walls 2 ft. above the springing line. These bars do not have sufficient section to take the total tensile stress at the crown, and, as will be seen, are supplemented by secondary bars which alternate with them and are bent to segmental curves with the ends radial, giving a firm anchorage in the concrete. Both sets of bars are $\frac{5}{8}$ in. in diameter and are 6 ins. apart on center at the crown. The $\frac{1}{2}$-in. longitudinal bars are wired to the transverse bars at intersections.

Comparative bids for the intercepting sewer of reinforced concrete as compared with the ordinary brick-lined concrete sewer showed a saving of from 19.7 to 22.4 per cent. of the cost in favor of reinforced concrete sewer.



Side Elevation.

Cross Section.

Fig. 499.—Invert and Invert Form for Sewer, at Beverly, Mass.

United Shoe Machinery Company's Sewer.—Figure 499 shows a unique form of sewer designed for use over a fill made on 5 or 6 ft. of mud in the bottom of the Bass River, Beverly, Mass. A line of piles 8 ft. centers was driven to support this sewer and keep it in line. The lower part of the sewer was designed as a reinforced beam 10 ins. square to span between adjacent piles. The beam and invert were constructed as a monolith. The molds were of 2-in. lumber in 8-ft. lengths, with a rib at each end and one in the middle. These were held in place by blocks on the sides of the piles. The earth formed the bottom of the form. After the outside form was in place the metal was placed, the concrete poured in and brought up to an inch or so below the floor line. Then the inside form was put in place and the concreting continued to the top of the invert. The forms used for molding the arch pieces are shown in Fig. 500. The arch ring was constructed in advance in 2-ft. lengths, and was reinforced with ¼-in. twisted steel rods spaced 12-ins. centers. These sections were set in cement mortar, completing the sewer.

Brooklyn, N. Y., Sewers.—Reinforced concrete was used in the

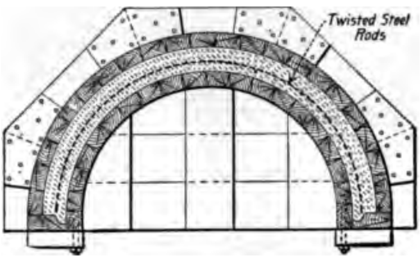


Fig. 500.—Arch Form for Sewer at Beverly, Mass.

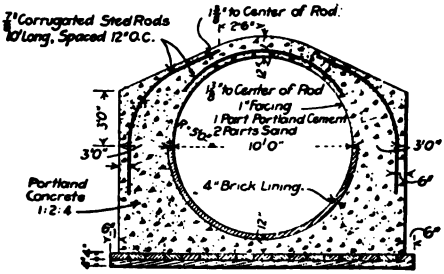
construction of 600 ft. of 10-ft. sewer on Foster Avenue, between East Nineteenth and East Twenty-first Streets, Brooklyn, N. Y. The depth of earth over the arch at this place ranged from 1 ft. 6 ins. to 3 ft.

Figure 501 shows the massive cross section used. The reinforcement consists of three ⅞-ins. corrugated steel bars arranged as shown on the drawing. The reinforcing rods were spaced 1 ft. centers along the sewer. No longitudinal rods were used. The concrete is 12 ins. thick at the crown, 3 ft. thick at the springing line, and 8 ins. at the invert. The sewer rests upon a timber foundation platform made of two layers of 4-in. plank.

Below the springing line the sewer is lined with a 4-in. ring of hard burned bricks. Above the springing line the arch is faced with 1 in. of mortar composed of 1 part Portland cement and 2 parts sand placed against the forms when the concrete is deposited. The outside of the top of the sewer is also plastered with 1 in. of mortar of the same mixture.

The method of construction used is as follows: After excavating a section the bottom of the trench was levelled and the foundation platform laid. On this the concrete for the section of the sewer below the springing lines was put in place, the sheet piling acted as a side form and was left in place.

When enough concrete had been deposited to bring its top surface up to a point slightly below the flow line, an inside form was put in place and held down by short posts placed under the trench braces. When the concrete in the haunch walls reached the proper elevation the side steel bars were put in



Cross-Section of 120-inch Concrete and Steel Sewer

Fig. 501.—Foster Avenue Sewer, at Brooklyn, N. Y.

position and held in place by strips of wood with notches cut at 1-ft. intervals. When the concrete was put in these strips were removed. The forms for the invert were left in 36 hours. When the concrete had reached a proper elevation the arch molds were put in place, the reinforcing bars for the crown put in position and blocked up at a proper distance from the forms by means of small wedges, which were removed as the concrete was brought up. The forms for the arches were left in place 48 hours. A 1:2:4 Portland cement concrete was used. The rods were bent to the proper curvature at the site. The lagging was of ⅞-in. planed lumber in narrow widths. Crude oil was smeared over the forms to prevent the adhesion of the concrete.

The South Bend, Ind., Sewer.—Over 5,900 lineal feet of reinforced concrete sewer have been recently constructed at South Bend, Ind. Two sizes were used, 72 and 66 ins. in diameter.

The sewer runs through a sandy soil having good drainage. Only the arch of the sewer is reinforced. The reinforcement consists of $\frac{3}{16} \times 1$-in. steel bands spaced 12 ins. centers, and placed 3 ins. inside the intrados of the arch. The arch ring is 9 ins. in thickness. The reinforcing bars are in three pieces; 1 piece on each side extends from 15 ins. below to 6 ins. above the

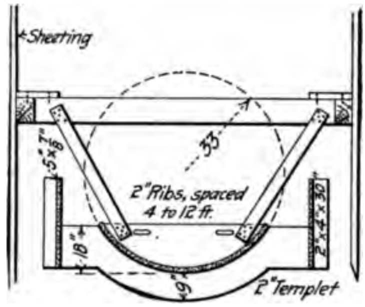


Fig. 502.—Invert Form, South Bend, Ind., Sewer.

springing line of the arch, with a piece in the arch joining these side pieces. The 3 pieces are fastened together with cotter pins. The arrangement of the rods and forms used are shown in Figs. 502-4.

The sewer was built in 12-ft. lengths. After the trench was

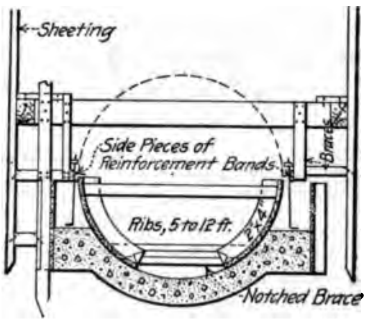


Fig. 503.—Side Forms, South Bend, Ind., Sewer.

graded 4 braces were nailed across the trench between the lowest ranges of the trench sheeting. A vertical row of lagging acts as a partial outside form for the bottom concrete. The template for the invert of the sewer barrel was suspended from the 4 cross braces as shown in Fig. 502. The concrete was carried up to the top of the template on the side, after which the template was removed. The side pieces of the reinforcement were then set and the side forms shown in Fig. 503 put in position. These sections extend up to the springing lines of the arch and were placed as soon as the invert template was removed. A notched brace at the bottom and a brace piece at the top hold these in place. The concrete is then carried up to the springing

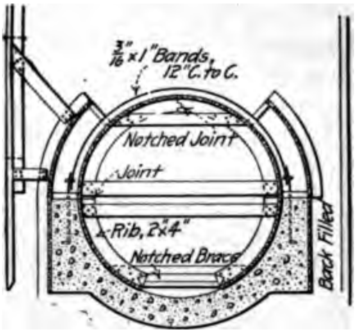


Fig. 504.—Arch Forms, South Bend, Ind., Sewer.

line as shown in Fig. 504, and the upper section of the forms put in place, the remaining reinforcements put in place and the concreting completed. Curves in the sewer were made in chords of arc 10 ft. long, an additional face being provided to overcome the friction at the angles between the straight chord sections. In this way expensive forms for curves in the sewer pipe were avoided. A 1 cement, 3 sand, and 6 gravel mixture was used for the invert and trench wall of the arch, while a 1 : 2 : 4 mixture was used for the arch ring. After the sewer barrel was finished it was coated with ½ in. of 1 to 1 cement mortar up to the springing lines of the arch.

The contract price for the construction of the 72-in. sewer, including excavation to an average depth of 18 ft. and back fill, was $9.75 per lin. ft., while the contract price for the 66-in. sewer was $9.50 per lin. ft.

Des Moines, Ia., Sewer.—Figure 505 shows two sections of circular sewer 7 ft. in diameter and flat sewer 5 × 10 ft. in sections, recently constructed at Des Moines, Ia. The flat section was used where only a limited clearance was available. The details of construction are clearly shown in the figures.

Special Moulds for Small Sewers.—A centering suitable for sewers and conduits of small size was used in the construction of a 30-in. composition sewer at Medford, Mass. The crown of this sewer, occupying about 120° of arc, was of brick and the remainder of the sewer of concrete. The concrete portion was constructed as a monolith. The forms (Fig. 506) were constructed in 10-ft. lengths and made in halves separating on a vertical line through their center, and were connected by clamps and held at a proper distance apart by iron dogs in the end

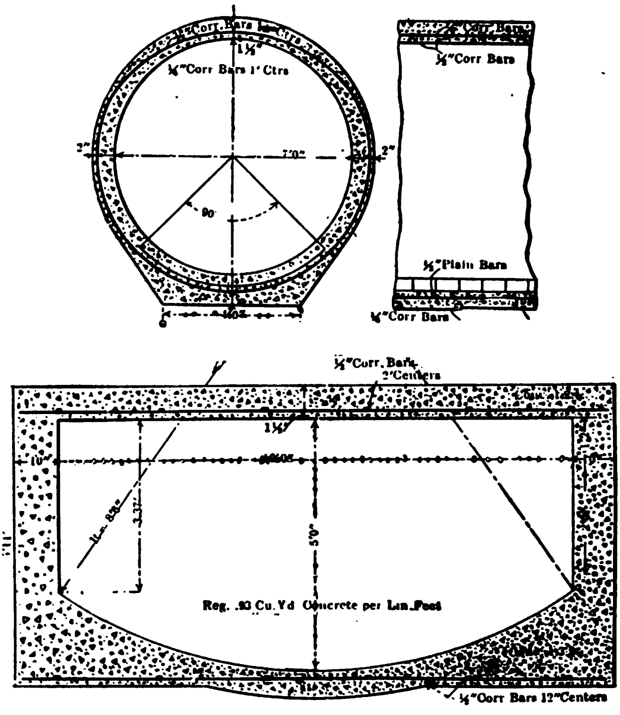


Fig. 505.—Sewer at Des Moines, Iowa.

ribs of each form. After smearing the forms as usual they were set in the trench and the concrete deposited and rammed. When it had partly set the dogs were removed and replaced by turnbuckles, which were slowly turned, so that the upper portions of the forms approached each other slightly, thereby separating them from the green concrete without injuring it. The forms were then withdrawn. The arch center was then put in place and the brickwork laid. These centers were also in 10-ft. lengths. The lagging was ⅞ in. thick by 1½ in. wide, with one edge beveled to make a tight surface. Ribs of 2-in. plank spaced 2 ft. centers supported the lagging. It is stated that the total cost of this sewer, exclusive of manholes, was $2.18 per lin. ft.

A novel patent sewer form adapted to the construction of medium and small sized sewers is shown in Fig. 507. This form consists of sheet steel plates bent to the required shape and held together by clips made by bending the edges of the plates to

Fig. 506.—Form for Sewer Invert at Medford, Mass.

the shape shown in the figure. These clips are held in place by filling the spaces inside them with paraffine or clay. The outside of the form is coated with grease or may be wrapped with paper or burlap in the usual manner. The trench is excavated to the size desired for the outside of the sewer and if necessary, outside forms of the usual type are employed. The bottom concrete and reinforcement, if any is used, put in place, the form put together, lowered in place and the remaining concrete deposited. When it is desired to remove the forms the paraffine is melted in some manner, or if clay is used, it is washed out and the form collapses and is removed.

Another patent steel form described in Engineering News, October 20, 1904, is shown in Fig. 508.

This form consists of a steel shell bent to the shape of the conduit desired and braced internally by turnbuckle rods hooked into suitable eyes and bearings. The shell is made in two parts, an upper part or center, and a lower part or invert form. Near its top edge on both sides the invert form is provided with hook-eyes which are headed through the shell with countersunk heads. The bottom edges of the arch have plates riveted on their inside, which lap past the edges of the invert form and have slots

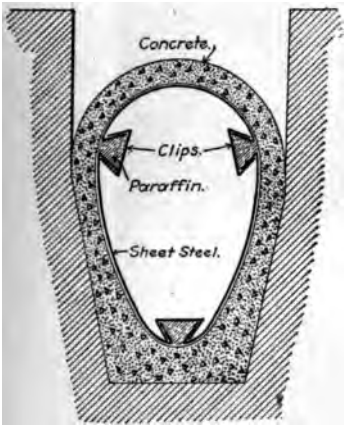


Fig. 507.—Steel Forms for Sewers.

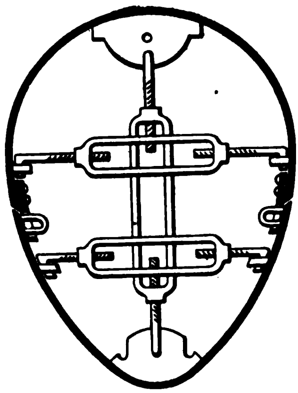


Fig. 508.—Blaw Collapsible Steel Centering for Sewers.

through which the hook-eyes pass. Cotters through the hook-eyes clamp the arch center and invert together. The turnbuckle rods brace the shell internally and preserve its shape under load. These forms are usually made in 5-ft. sections. The Blaw Collapsible Steel Centering Co., Pittsburg, Pa., control the patents and manufacture this form.

A patent form used in the construction of a 5-ft. egg-shaped sewer at Washington, D. C., is shown in Fig. 509.

Forms of similar construction have been successfully used for building sewers as small as 8 ins. The centering consists of a wooden form made in two parts firmly attached together when in use, about which is wrapped a thin steel strip about 6 ins. in width, forming a continuous coil or covering upon the surface of the forms. After the strip is in place it is smeared with oil and the form is lowered into the trench, the concrete put in, the wooden form collapsed and removed. The steel strip, which is usually of No. 20 gauge metal, is left in place to support the concrete until it has set, when it is removed by pulling on one end of the strip. The centering, usually in 16-ft. lengths, is made of

Fig 509.—Spirally Wrapped Sheet Steel Form for Sewers.

lagging nailed to ribs spaced about 2-ft. centers. Wedge timbers marked A and B in the drawing, are provided to keep the forms the proper distance apart. The two halves are locked together, by latches on the outside of the end ribs. A hole C is provided in the centering through which a square gudgeon-timber is passed lengthwise of the centering. The ends of this timber are rounded and the form is mounted on bearings carried by trestles. The sheet steel covering is wound upon the form by revolving it on its bearings. When it is desired to collapse the centering the wedge timbers are driven in and the upper and lower parts close together and are then withdrawn.

Fig. 510.—The Ransome Concrete Pipe Mold.

The Ransome Pipe Mold has been successfully used for the construction of small concrete pipe varying from 4 ins. to 24 ins. in diameter. This device consists of a form made of sheet steel, having an inner core 10 ft. in length. One end of this core is surrounded by a short steel shield that serves as the outer form of the cement pipe. The mortar for the pipe is packed in between the inner core and this outer shield by a man who uses a small rammer for this purpose. Fig. 510 shows this mold being used in the construction of pipe. A man standing in the foreground keeps moving the mold forward slowly by means of a lever. This lever is provided with a dog operating in a clutch, which rotates a small drum on which a wire is wound. The wire rope is anchored into a dead man in the trench ahead. As the mold is moved forward it leaves behind it the cement pipe, which is still green. The cement mortar is mixed with a small amount of water, so that it possesses sufficient cohesion to hold together when unsupported by the core.

To protect the pipe until it hardens it has been found advisable to pack a little earth around its sides and over the top. This is done by a third man in the trench, who packs the filling upon the part of the pipe where it is still supported by the core.

It has been found that the pipe does not cave in unless a heavy body falls upon it before the cement has hardened. It is stated that a pipe does not break down under its own weight even when made as large as three feet in diameter.

When it is desired to connect a branch pipe to the main one, a hole is cut in the side of the green pipe before the core has been pulled ahead. A branch of the proper pattern is shoved up tightly against the pipe, and the collar of the branch is plastered with cement mortar, producing a strong watertight connection.

The itemized cost for the construction of an 8-in. cement pipe built at Despatch, N. Y., for the Foster-Armstrong Piano Co., is given as follows:

6 men at $1.70 per day, 10 hours	$10.20
1 foreman	2.00
3 barrels cement at $1.25	3.75
3.3 cubic yards of sand at $0.85	2.80
Water	0.15
Total for 300 lineal feet	$18.90

This is equivalent to 6.3 cents per lin. ft. of pipe. It is found that a gang of six men would average about 300 lin. ft. per day.

In the construction of a 12-in. pipe, the cost was as follows:

7 men at \$1.70 per day	\$11.90
1 foreman	2.20
13 barrels of cement at \$1.33	17.30
12 cubic yards of fine gravel, at \$1.80	9.60
Total for 400 lineal feet of pipe	\$41.00

This is equivalent to 10¼ cents per ft.

It is stated that the cost of vitrified clay pipe, 8 ins. in diameter, under like conditions, is about 17½ cents, and the cost of 12-in. vitrified pipe will be about 35 cents, showing a great saving when cement pipe is used.

In making estimates on the cost of cement pipe, however, the cost of the cement and the labor items should be carefully considered when comparing it with some other form of pipe. But under almost all conditions it will be found that cement pipe can be constructed at great saving over other forms.

As to durability, the cement pipe is as strong, or stronger, than vitrified clay pipe. The pipe which is constructed continuously is desirable on account of the lack of joints, there being no joints for leakage unless the pipe is injured in some manner. This pipe has been successfully used for sewers, water mains, etc.

CHAPTER XXVIII.

TANK AND RESERVOIR CONSTRUCTION.

As has been stated, the first application of reinforced concrete by Joseph Monier was in the construction of tubs and basins for use in horticulture. Monier later became bolder and used this new material in the construction of water tanks and reservoirs, some of large size. The use became greatly extended and reinforced concrete is to-day almost exclusively used for tanks, reservoirs, etc., in Europe.

The many good qualities of reinforced concrete make it an especially valuable material for the construction of large and small tanks, both rectangular and circular—for reservoir, storage bins for cement, coal, grain, sand, etc. It is also used as a lining for reservoirs constructed of masonry or earth, and for reservoir roofs.

Tanks and reservoirs may be buried under the ground, placed directly upon the ground, or supported at any desired elevation upon towers of steel, masonry or reinforced concrete. The position of the tank modifies to some extent the structural details adopted, as does the shape, whether circular or rectangular. The general system of reinforcement consists, however, of a network of rods, the size and spacing of the rods varying with the loads to be carried. When rectangular tanks are constructed, the sides consist of reinforced slabs, sometimes strengthened with ribs or beams. The horizontal rods of circular tanks and reservoirs are spliced by lapping or welded into hoops. The tank bottoms may or may not be reinforced, depending upon the nature of the subsoil; when they rest upon the ground they are usually approximately flat, only having enough slope to drain them properly. When tanks are elevated, the bottoms are either of spherical or conical shape, usually with the convex surface upwards, although it is sometimes placed downwards.

The horizontal rings or hoops forming the reinforcement for circular tanks are placed close together at the bottom and spacing gradually increased toward the top, while the vertical rods are spaced uniformly around the tank, the two sets being wired together at intersections. In rectangular tanks horizontal and vertical rods are also used and spaced similarly to those of circular tanks. At the corners formed by adjacent sides the horizontal rods are usually bent around to make the reinforcement continuous, thereby making the tank as strong at this point as at any point in the side wall. The vertical rods and bottom rods may be bent in the same manner to form a solid junction between the side walls and tank bottom.

Roofs for tanks may be either flat or ribbed slabs, spherical or parabolic arches, spherical arches being preferred when any great load is to be carried on the cover. For flat, ribbed slabs and parabolic arches the usual types of reinforcement are used, while for spherical arches concentric rings, with radial rods wired together, give a satisfactory means of reinforcement. The Monier type of reinforcement, with varying spacing of the rods, is almost universally employed in tank and reservoir construction.

For small rectangular tanks the arrangement of the reinforcement is quite simple, a Monier mesh being placed in each side and the bottom, with sufficient lap at side wall and side and bottom wall junctions. Electrically welded wire, lock woven fabric and expanded metal are easily put in position and give a most satisfactory reinforcement. When tanks of somewhat greater size are used and it is desired to put a roof over them, girders are used with the roof slab.

Figure 511 shows a tank of the latter type constructed for the Pittsburg Lamp, Brass & Gas Company. This tank is 17 ft. wide, 65 ft. long and 11 ft. 6 ins. deep. It is located in a court between two buildings and it was found necessary to take care of the foundations of the walls of these buildings. To do this the side vertical beams shown on the drawings were used, although these are not an essential part of the tank proper.

Milford, O., Standpipe.—The standpipe at Milford, Ohio, is an example of the use of reinforced concrete to replace the usual standpipe built of steel. This structure is 81 ft. high from base to under side of roof. The roof is dome-shaped, with a rise of 3 ft., making a total height of 84 ft. The outside diameter is 15 ft. 6 ins. The shell at the base has a thickness of 9 ins. This thickness is maintained to a height of 30 ft., where it reduces to 7 ins. and again at the height of 55 ft. to 5 ins. The reduction in thickness is made on the inside of the pipe. Both an inside and outside ladder is provided.

The foundation is octagonal, with an inscribed diameter of 20 ft., and is 6 ft. deep. It was constructed of concrete composed of 1 part of cement to 7 parts of gravel, while the concrete for the standpipe proper was 1 part of cement to 3 parts clean, sharp sand. On top of the foundation concrete 1 × 1 × 1/8 T-bars were laid radiating from the center to within 6 ins. of the outer edge. The shell was started directly on these T-bars and, after being

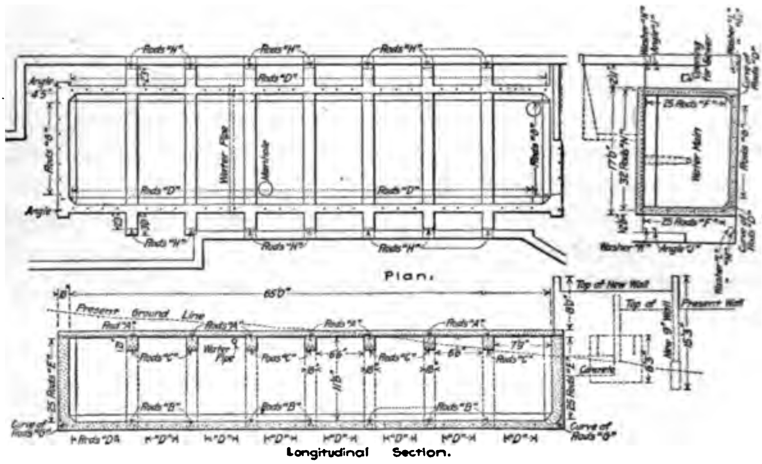


Fig. 511.—Rectangular Tank, Pittsburg Lamp, Brass and Gas Co.'s Works.

carried up a sufficient distance, the base outside the shell was covered with concrete 16 ins. deep, and the base inside the shell was covered with a 6-in. layer of 1 cement and 3 sand mortar.

The shell is reinforced by a network composed of verticals spaced 18 ins. apart around the structure and horizontal rings, six to the foot for 30 ft., then five to the foot for 25 ft., and then four to the foot for the remainder of the height. Both vertical and horizontal reinforcements consist of 1 × 1 × 1/8-in. T-bars connected at intersections by clamps stamped out of sheet metal, similar in form to the Streeter clip which is used for connecting structural steel work. The reinforcement was located 3 ins. inside the outside face of the shell.

The forms used in the construction of this standpipe were made of 1⅛ flooring 3 ins. wide and 3 ft. long for staves, nailed to 4 × 4-in. circular ribs. The topmost rib extended 1 in. above the top of the staves, so as to form a rabbet to receive the bottom of the next form. Three sets of forms were used, each consisting of an outer and inner form and each divided into eight sections for convenience in handling. The sections of each ring of the outside forms were held together with latches, and those of the inside forms were bolted together. An average height of about 5 ft. a day was constructed. This standpipe was designed and constructed under the supervision of Mr. J. L. H. Barr, of Batavia, O.

The Fort Revere Tower.—A good example of a standpipe and

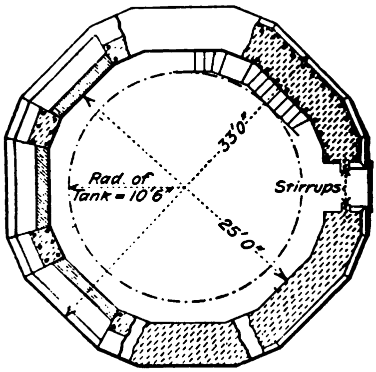


Fig. 512.—Horizontal Section of Base of Fort Revere Water Tower.

water tower construction of Hennebique design is that constructed at Fort Revere, Boston Harbor, Mass. The original design was for a masonry tower and steel standpipe, but a bid for the work in reinforced concrete by the Hennebique Construction Co. that was 30 per cent. lower than any bid on the original design led to the adoption of reinforced concrete.

The tower is octagonal in form and consists of eight reinforced concrete piers resting upon a reinforced moulded base about 12 ft. high. The filling between the piers consists of brick. The pier support a reinforced concrete floor at an elevation of about 2 ft. above the top of the standpipe. This floor is a ribbed slab 3 ins. thick, having ribs 6 ins. wide and 12 ins. deep. Above this floor is an observatory with a wooden roof. The total height of the structure is about 93 ft. The piers are reinforced with six ¾-in. rods. Horizontal and vertical sections of the base are shown in Figs. 512 and 513, while a vertical section of the tower proper is shown in Fig. 514. The interior diameter of the tower is about 25 ft., while the diameter of the standpipe is 20 ft., the space between the two being occupied by a spiral staircase. The standpipe has a height of 50 ft. The shell is 7¼ ins. thick at the bottom and 4½ ins. at the top. The bottom of the tank is 4 ins. thick. Figs. 515 and 516 show plan and section of the bottom and section of the side wall at the bottom. Both the wall and floor are coated on the inside with 1 in. of 1 to 1 mortar to prevent leakage. The arrangement of the reinforcement and method

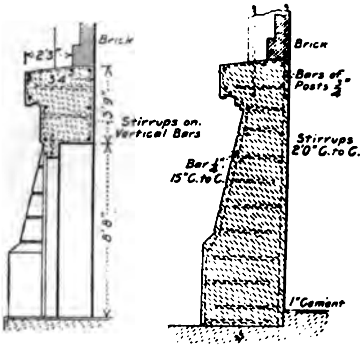


Fig. 513.—Vertical Section of Base of Fort Revere Water Tower.

of bonding together the walls and bottom are shown in the figures.

The wall is reinforced with two sets of vertical and horizontal rods. The upright rods, which are $^5/_{16}$-in. in diameter, are 2 ins. apart transversely and are staggered, the rods in each set being spaced about 16 ins. apart circumferentially, making one vertical rod every 8 ins. about the circumference of the shell. The two sets of horizontal bars each encircle one of the sets of vertical bars and are made of ½-in. diameter rods with welded joints for the lower two-thirds of the tank and ⅜-in. rods with lapped joints wired together for the upper one-third. The vertical spacing of the hoops increases as the height of the shell increases. For the ½-in. hoops there are 23 spaces of 1¾ ins.; 41 spaces of 2 ins.; 34 spaces of 2½ ins.; 22 spaces of 3 ins.; 13 spaces of 3½ ins., and 23 spaces of 3¾ ins. For the ⅜-in. hoops there are 9 spaces

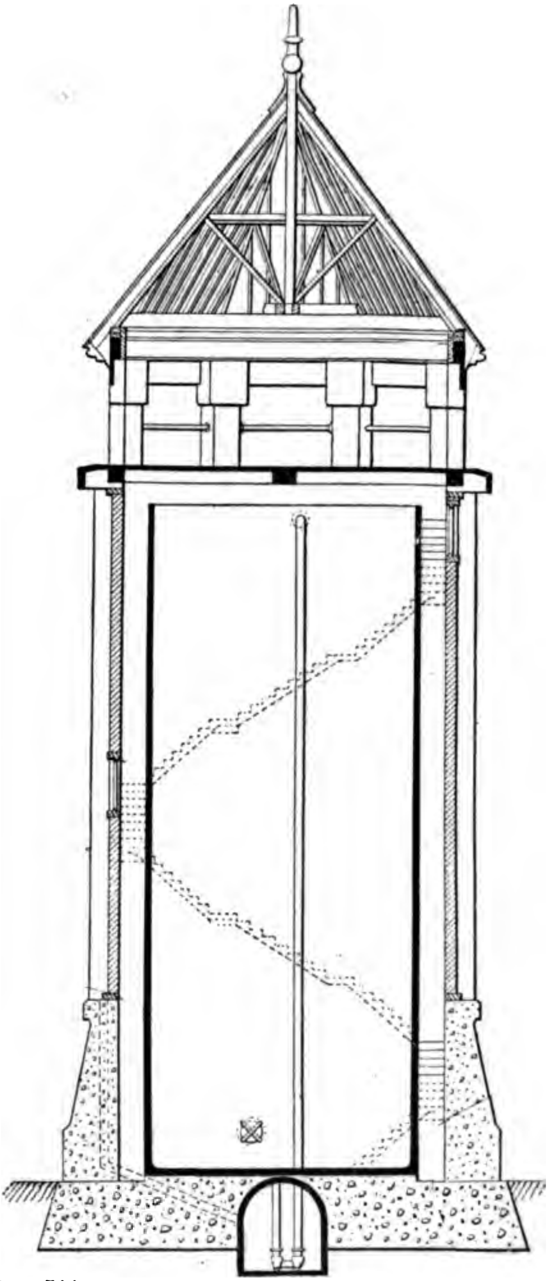


Fig. 514.—Vertical Section Fort Revere Water Tower.

of 3-in.; 6 spaces of $3\frac{1}{2}$ ins., and 6 spaces of $3\frac{3}{4}$ ins., the inner and outer hoops at each level up to this elevation being in the same horizontal plane. For the remaining 16 ft., the two sets of

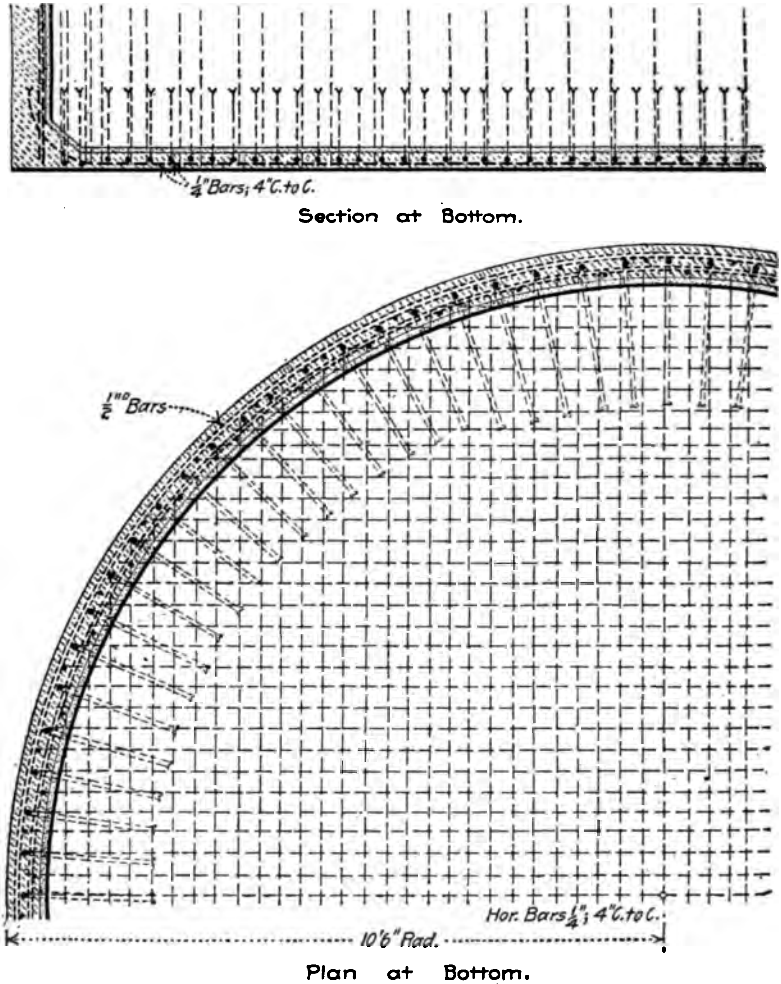


Fig. 515.—Plan and Section of Buttress, Fort Revere Water Tower Tank.

hoops are staggered and the spacing between adjacent hoops varies from 2 to $7\frac{1}{2}$ ins.

The bottom of the standpipe is reinforced with two sets of $\frac{1}{4}$-in. rods, spaced 4 ins. centers, and crossing one another at right angles. The rods are bent up at their ends, and extend into the wall for a height of about 12 ins. The junction of the walls with the bottom is further reinforced with a set of ⅜-in. rods extending about 20 ins. radially into the floor slab and running up to a height of about 24 ins. in the wall. These rods are placed in the center of the thickness of the wall and floor slabs and are bent at an angle of 135°, passing near the inner surface of the concrete, which is thickened at this point. A circular hoop is placed at this angle and stirrups of 1 × ¼-in. steel, 7 ins. long, spaced 8 ins. centers, placed about the ⅜-in. anchor rods, tie the whole firmly together.

The East Orange Reservoir.—The reservoir for the East Orange Water Supply is 139 × 240 ft. in plan on the inside and has a capacity of 5,000,000 gallons with a depth of water of 20 ft. The

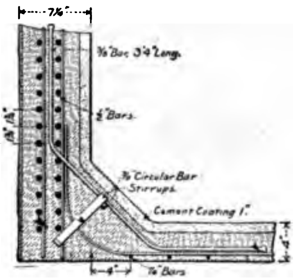
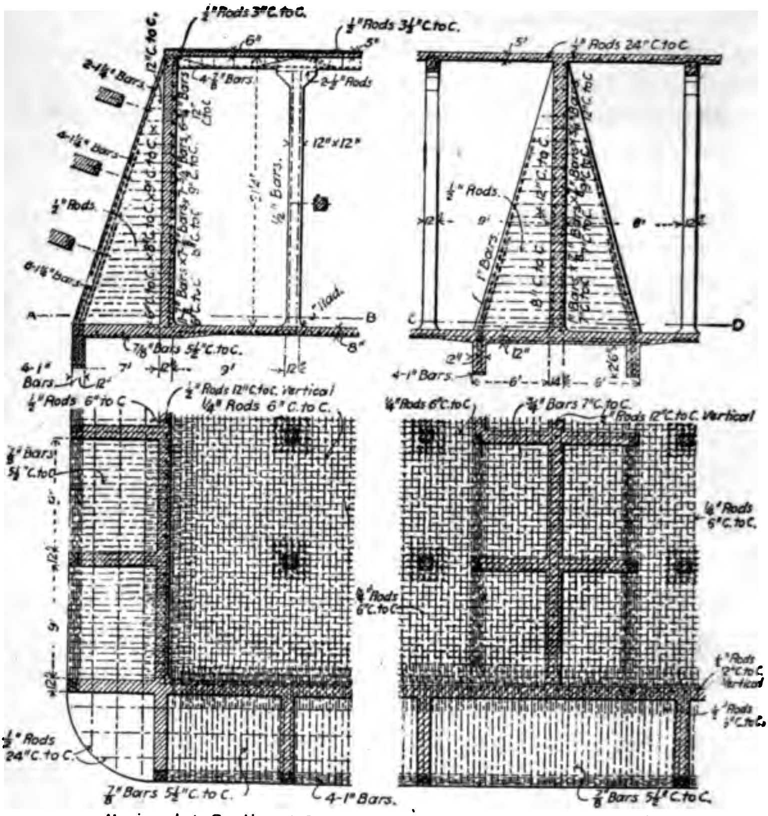


Fig. 516.—Section of Tank at Bottom Corner, Fort Revere Water Tower.

interior height from floor to roof is 21 ft. 4 ins. The reservoir is divided by a partition wall into two equal and nearly square basins so arranged that each is independent of the other and one may be shut off and emptied while the other is in use.

The exterior walls are formed of a reinforced slab 12 ins. in thickness, braced by counterforts or buttresses spaced 10 ft. centers. The buttresses are 12 ins. thick and 7 ft. wide at the bottom and were constructed as integral parts of the wall monolith. The reinforcement for the walls and buttresses is given in Fig. 517. The division wall is 14 ins. thick and has buttresses 6 ft. wide on both sides, also spaced 10 ft. centers.

The floor concrete is 8 ins. thick, except under the wall buttresses, where it is 12 ins. thick. The floor reinforcement consists of ¼-in. corrugated bars, spaced 6 ins. centers. The floor extends 7 feet beyond the walls on all four sides. Under the outer edge on each side there is a beam 12 ins. wide and 30 ins. deep formed in a trench. These beams are reinforced by four 1-in. rods placed 1 in. above their bottoms. Similar beams are constructed beneath the floor at the toes of the division wall bottoms.



Horizontal Section A-B.
Wall, Buttresses, Columns, Beams and Flooring.

Horizontal Section C-D.
Center Wall.

Fig. 517.—Details of Walls and Buttress, East Orange Reservoir.

One-half inch rods 6 in. centers and 4 ft. long were placed under the exterior walls and columns.

The roof is a flat concrete slab 6 ins. thick for 10 ft. from the exterior walls, and 5 ins. thick over the remaining area, and is reinforced with ½-in. steel bars spaced 24 ins. centers longitudinally and 3½ and 3 ins. transversely in the 5 and 6-in. slabs respectively. This slab is supported by the walls and columns, spaced 10 ft. on centers, carrying girders 12 ins. wide and 16 ins. deep below the under side of the roof slab. The columns are 12 ins. square and are reinforced with four ½-in. vertical bars tied together in the usual manner with $\frac{3}{16}$-in. tie wires. The columns are connected to the girders with a corbelling 3 ft. long. The girders are reinforced with four ⅞-in. bars, two being straight and two bent, as shown, with four additional ⅞-in. rods 5 ft. long over the top of each column. Stirrups are also used to reinforce the girders against shear.

Expansion joints were formed in the exterior and division walls of the reservoir at intervals of about 50 ft. by inserting a plate of steel ¼-in. thick and 6 ins. wide 21 ft. long, covered with two sheets of lead ⅛-in. thick bent in U-shape and fitting tightly to the steel plate. These were also used as division planes at which the work was stopped and started.

The reservoir is waterproofed by covering the water sides of the floor and walls with a 1-in. coating of 1 : 2 Portland cement mortar mixed with a solution of light soft soap in the proportion of 1¼ lbs. to 15 gallons of water and having 3 lbs. of powdered alum incorporated with each bag of cement. This mortar coating was deposited as the concrete was put in place.

Fort Meade Reservoir.—An excellent example of reinforced concrete reservoir construction is the 500,000-gallon reservoir recently constructed for the United States Cavalry Post at Fort Meade, S. D. This reservoir is built on a hill 200 ft. above the post and serves as a distributing reservoir.

The reservoir is divided into two compartments, each 50 ft. by 60 ft. and 16 ft. in height from top of floor to under side of roof slab. The corners are rounded and a division wall separates the two compartments, each of which was designed to hold 250,000 gallons when filled to top of overflow pipes.

The bottom of the reservoir floor was fixed at about 9 ft. below the ground surface. This required considerable excavation to secure the proper depth. The material encountered in the excavation consisted mainly of coarse gravel, mixed with fine sand and clay. This afforded an excellent foundation when confined, as it was, at the bottom of a deep cutting. After the excavation was completed to sub-grade the entire bottom was rolled with a heavy roller, thoroughly compacting the material. The footing excavations were afterward dug to the proper size and depth without disturbing the adjacent materials.

The floor is 10 ins. thick and is reinforced with ¾-in. longitudinal and transverse rods spaced 12 ins. centers. The floor division and side walls are designed to resist the pressure of the earth when either or both are empty as well as the pressure resulting when either or both compartments are filled. The roof is designed for an ultimate load of 800 lbs. per sq. ft., the actual

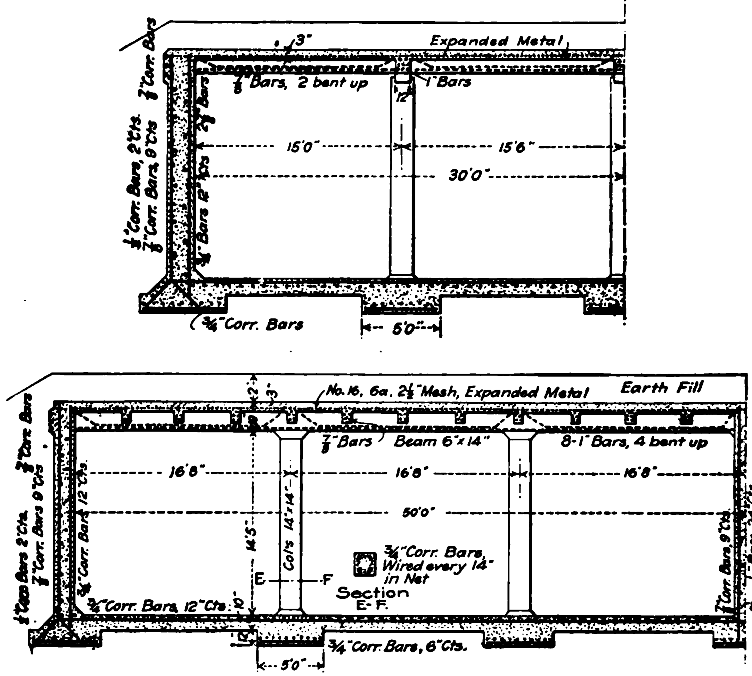


Fig. 518.—Reservoir at Fort Meade, South Dakota.

load consisting of 12 ft. of earth fill and 100 lbs. snow load. The concrete footings are 5 ft. square and 12 ins. deep below the bottom of the regular floor. The wall footings are 5 ft. wide, the full length of the walls. All footings are proportioned for a uniform pressure for the sub-soil of 1.5 tons per sq. ft. As shown in the section (Fig. 518), the footings are reinforced with a grille of ¾-in. corrugated bars. No evidence of settlement was observed.

The columns are 14 ins. square and are reinforced against flexure for four ¾-in. bars, one placed in each corner and the four tied together with iron wire at intervals not greater than the diameter of the column. The columns are designed to carry the total load with an average unit stress of 500 lbs. per sq. in. The exterior walls, as well as the division wall, are considered as vertical beams and reinforced accordingly with vertical bars, while horizontal bars are provided to care for temperature stresses.

Each compartment is divided into three transverse bays, and three longitudinal bays extend through both compartments. The roof slab and beams are carried directly by three longitudinal 12 × 22-in. girders, which rest directly on the columns and side and division walls. The girders are considered as acting as T-beams. The girder reinforcement consists of eight 1-in. corrugated bars placed in two rows near the bottom of the beam. The upper bars are bent up over the supports to better resist the shearing stresses. The four bottom rods extend the full length of the beam in one plane. The roof beams are 6 × 14 ins. in cross-section, and are each reinforced with four ⅞-in. bars. The roof slab is 3 ins. thick and is reinforced with 2½-in mesh, No. 16 expanded metal.

The concrete was a 1:2:4 mixture with all stone crushed to a size no larger than a ¾-in. cube, crusher-run stone being used. An effort was made to secure as dense a concrete as possible, and a very wet mixture was used. The concrete was not tamped but was spaded to allow the air bubbles to escape. A specially designed spading bar was used with a blade or paddle 6 × 3 × ¾-in., mounted on a ¾-in. round iron handle 5 ft. long.

The bottom excavation for the reservoir was made practically level and the concrete placed directly upon it. The excavation for the wall and column footings were made slightly larger than the footing dimensions and a 2-in. plank placed on edge around their sides before placing the concrete. The lagging for the wall forms consisted of 2-in. plank dressed on one side and nailed to 4 × 4-in. uprights 4 ft. on centers in pairs on each side of the wall. The lagging in the forms for the round corners of the outside wall were thin boards bent to the required curve and nailed to posts 2 ft. apart. The posts were held in place by outside struts and by connecting wires passing through the wall space between the edges of adjacent planks.

The column forms were made of three side pieces of 2-in. timber extending from the floor to the girder forms, the fourth side being left open to receive the concrete. These forms were braced at the bottom by struts extending on all four sides to a firm bearing. At the top they were held in place by braces extending to adjacent columns and to the walls.

The girder and beam forms were open troughs of the required dimensions, made of 2-in. plank with the smooth side in. For the roof slab a close floor of 2-in. plank with the smooth side upward was used. The slab centering and the girder and beam forms were supported by posts resting upon the floor below. For the footings under the walls and columns a thin layer of cement was first spread over the bottom and the lower reinforcing bars properly spaced were pressed into the concrete to the required depth. The upper bars were then placed crosswise over the lower ones and covered with concrete, care being taken not to displace the bars in placing the concrete. In placing the floor reinforcement practically the same method of procedure was followed. Before concreting the walls and columns the reinforcement was erected in place forming a steel skeleton, around which the concrete was placed. The four column reinforcing bars were held at proper distances apart by wooden plates, through which holes were bored to receive the bars. These plates were moved up out of the way as the concreting progressed. The column bars were wired together every 14 ins. in height, the wiring being done ahead of the concreting.

The vertical bars in the walls were spaced by means of planks, forming templates through which holes had been bored at proper distances apart to receive the bars. Each row of bars was spaced by an independent template that was moved upward as the work progressed and that was kept high enough to allow room below for placing and tamping the concrete. The horizontal bars were wired to the vertical bars, each bar being secured in its proper position as the concrete was put in. The girder reinforcing bars were blocked up and the blocking removed as the concrete was put in place.

The expanded metal used for reinforcing the floor slab was laid directly upon the centering and after being covered by some concrete roughly shoveled over was raised slightly by means of hand hooks to insure the metal being slightly above the bottom of the slab and entirely surrounded by concrete.

The walls were built up as near as possible to a uniform height all around, no part being at any time carried higher than 4 ft. above any adjacent part. At places where the work was stopped for the night or temporarily, stop boards were used to square up the ends of the concrete. These boards were placed vertically and were not over 4 ft. in height and extended the full width of the wall. These stop boards consisted of a 2-in. plank, to the center of which was nailed a 4 x 4-in. cleat. The cleat formed a recess in the concrete into which the fresh concrete flowed, forming a tenon when the work was continued. This made a firm bond between the new and the old concrete. Care was taken in all cases to brush off all loose materials from the old concrete and wash over its surface with cement grout before depositing fresh concrete.

Besides using great care to get a dense concrete, the interior surface of the reservoir was coated over with a coating of 1 cement to 7 sand mortar. This was put on in two coats and the whole brushed over with a coat of grout.

In this work the column and wall forms were usually removed from 7 to 10 days after the concrete was deposited. The girder and roof forms and centers were usually left in place from two to three weeks.

The Bloomington, Ill., Reservoir.—A reinforced concrete reservoir of 10,000,000 gallons capacity was recently constructed at Bloomington, Ill. This reservoir is 300 ft. in diameter and has side walls 15 ft. high, vertical on the back, and with a batter of 2 ft. on the front face. Figure 519 shows a section of the wall, which is 1 ft. thick at the top and 3 ft. thick at the bottom, and is built with a footing 10 ft. wide. This footing has a toe on the inner side 4 ft. deep below the bottom of the reservoir and 2 ft. wide. It is reinforced near the bottom with transverse ¾-in. corrugated bars placed 9 ins. centers. Every fourth bar is 10 ft. long and is bent down into the concrete toe of the footing. The other bars are 4 ft. long and have one end just inside the back of the footing. The footing is also reinforced near its upper surface on the water side with ¾-in. bars placed 6 ins. centers. These bars are alternately 4 and 6 ft. long, and all extend 18 ins. into the concrete under the wall.

The wall proper is reinforced with vertical ¾-in. bars near its inner face. These bars extend down into the footing and their spacing is reduced from 4 to 6 and then to 12 ins. on centers

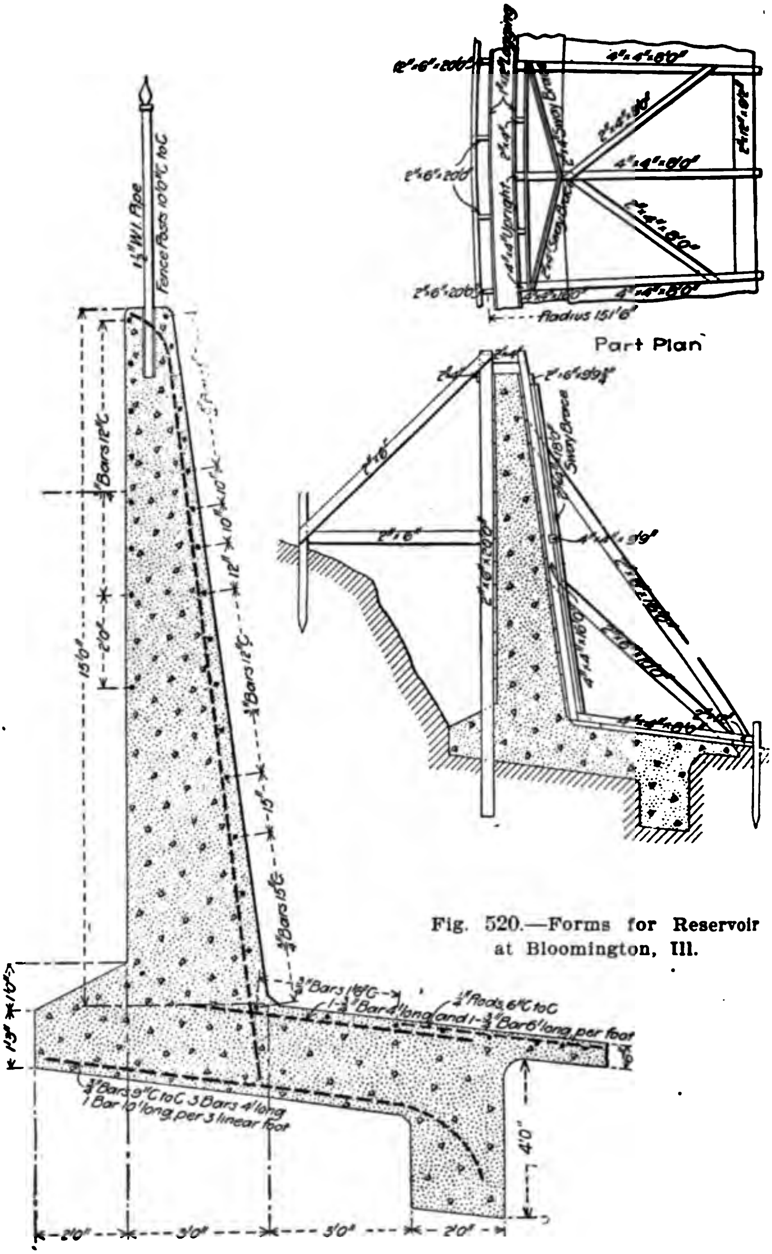


Fig. 520.—Forms for Reservoir at Bloomington, Ill.

Fig. 519.—Section of Wall for Reservoir at Bloomington, Ill.

from the bottom of the wall to the top. The wall is also reinforced for temperature stresses near both faces by ¾-in. horizontal bars. These are placed varying distances apart, depending on the liability of extreme changes of temperature, and on the back face are omitted below the frost line. The wall was built entirely without expansion joints.

The surface of the bottom of this reservoir is a segment of a sphere the depth of the reservoir, varying from 15 ft. at the wall to 25 ft. at the middle. The floor is 6 ins. thick and is reinforced with a lattice work of ¼-in. round rods, spaced 6 ins. centers in both directions.

The forms used in the construction of this reservoir were built in sections. Figure 520 shows the detail of a single section. The forms consisted of planks for lagging, nailed to vertical posts, which were accurately set and firmly braced. The forms were made in lengths equal to one one-hundredth of the circumference of the reservoir. Twenty-one of these form sections were built and all set up at once. The wall when started was built continuously in both directions from the starting point. The lagging consisted of 1-in. boards and was nailed to the vertical posts and was carried up just ahead of the concrete filling.

The footing was built without forms up to its junction with the wall proper. A layer of concrete 2½ ins. thick was first laid in the footing trench, the lower reinforcing bars put in place, the vertical bars set in position and the concrete filled in up to the level of the top layer of reinforcing bars; these were then put in place and the footing completed. The wall forms were then put in position as shown in the figure and the concreting continued to the top.

A facing of gravel concrete, made of 1 part cement to 4 parts fine gravel, with no pebbles larger than 1 in., was placed on the front face of the wall.

A sheet-iron form similar to that described on page 118 was used for depositing this facing. Both faces of the wall were painted with a 1 to 1 mixture of cement and sand; the inner face was also painted with a 1 to 1 mixture of waterproof Star Stettin Portland cement and sand.

The Failure of a Reinforced Concrete Reservoir Covering at Madrid, Spain.—The use of groined, parabolic and segmental arches in reinforced concrete for reservoir covers leads to extremely thin sections and a great saving in materials. Great care, however, should be used in the design when such thin sections are to be used, and precautions taken to avoid types of construction where the strength of any one part depends upon that of every other part, for if local failure takes place it may be followed by a collapse of the whole structure. Care should be taken to brace and stiffen the cover in both transverse and longitudinal directions, and extremely slender sections in columns, beams and arch

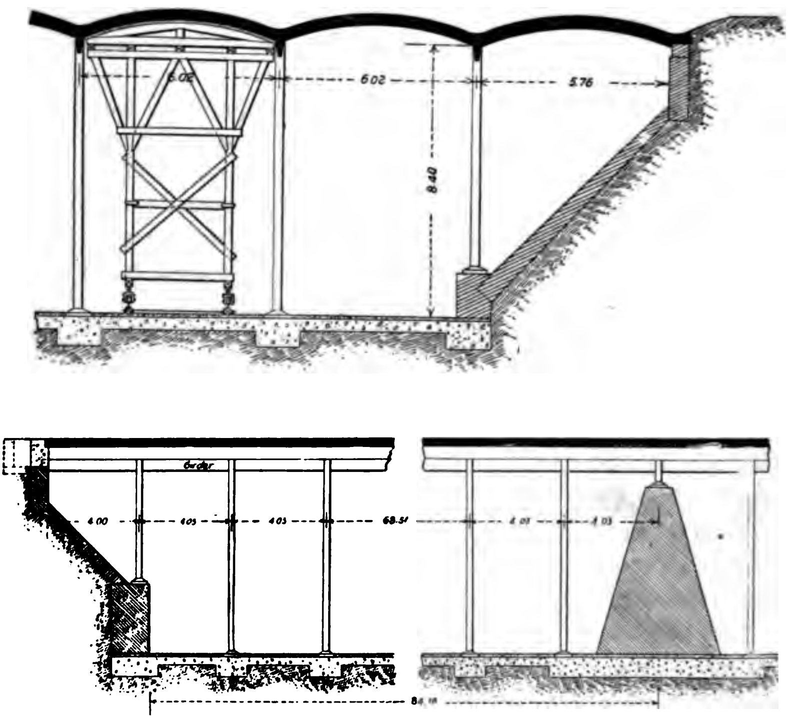


Fig. 521.—Plan and Sections of Reservoir at Madrid, Spain.

covering avoided. Temperature stresses also should not be overlooked. The failure of the Madrid reservoir covering will serve to illustrate the features of bad design to be avoided.

The Madrid reservoir has a capacity of about 106 million gallons, and was divided into four compartments of about 210 x 85 meters each, with a depth of 6.65 meters (about 690 x 280 x 22 ft.). Transverse and longitudinal sections of the reservoir are shown in Figure 521. The columns supporting the roof were

0.25 × 0.25 meters (9.84 × 9.84 ins.), and 8.4 meters (27 ft. 6¾ ins.) high to the springing line of the arches, giving a ratio of diameter to length of 1 to 34. The columns were spaced 6.02 × 4.03 meters (about 20 × 13 ft.) apart from center to center. These columns rested upon but were not rigidly attached to reinforced concrete bases. The columns were reinforced with four rods about ⅝-in. in diameter, and rested upon flats buried about 3 ins. in the reinforced concrete bases.

The covering consisted of parabolic segmental arches having a span of 5.77 m. (19 ft.), and a rise of 0.58 m. (2 ft. 2⅜-in.), and had a thickness at the crown of .05 m. (2 ins.). Parallel with the arches the columns are connected below the springing line by a reinforced concrete beam 0.5 m. deep (19⅝ ins.), having a single tensile reinforcing rod 37 mm. (about 1½ ins.) and 3 compression rods 15 mm. (about ⅝ in.) in diameter. These upper and lower bars are connected by vertical and diagonal rods 4, 5 and 6 mm. in diameter. No ties were used to connect the columns in a direction at right angles to these girders. The arch reinforcement consisted of rods about ½ in. in diameter tied together with $\frac{3}{16}$ in. diameter rods. A 1:4½ or 1:5 Portland cement concrete was used. The details of the roof construction are shown in Fig. 522.

The roof of one section of the reservoir had been completed and was tested by loading one line of arches, a width of 4 meters, on April 7, 1905, with 0.8 meters of sand; over 250 lbs. per sq. ft. On April 9 the covering collapsed killing 30 men and wounding 60 others. The collapse was sudden and without warning. The collapse was probably due to several causes. The test load was undoubtedly excessive for such a slender arch covering, probably causing the loaded crown to sink and the adjacent panels to rise. The extremely slender columns used, the absence of any rigid attachment at the base and entire absence of all transverse bracing probably all contributed to the failure of the roof covering. The temperature stress due to hot weather undoubtedly was a contributing cause to the collapse. It is stated that the excessive expansion of the concrete in a line of girders in one of the unfinished sections forced the beams out of line 2 ft. or more a few days after the collapse of the finished section. This deflection became so great that the columns and line of girders collapsed.

The stability of this structure depended upon every part maintaining perfect rigidity vertically and horizontally without the slightest deviation, for as soon as any movement took place in any single member the others were caused to move in their turn, bringing on a general collapse. It would appear from the above data that this structure was not carefully designed, many of

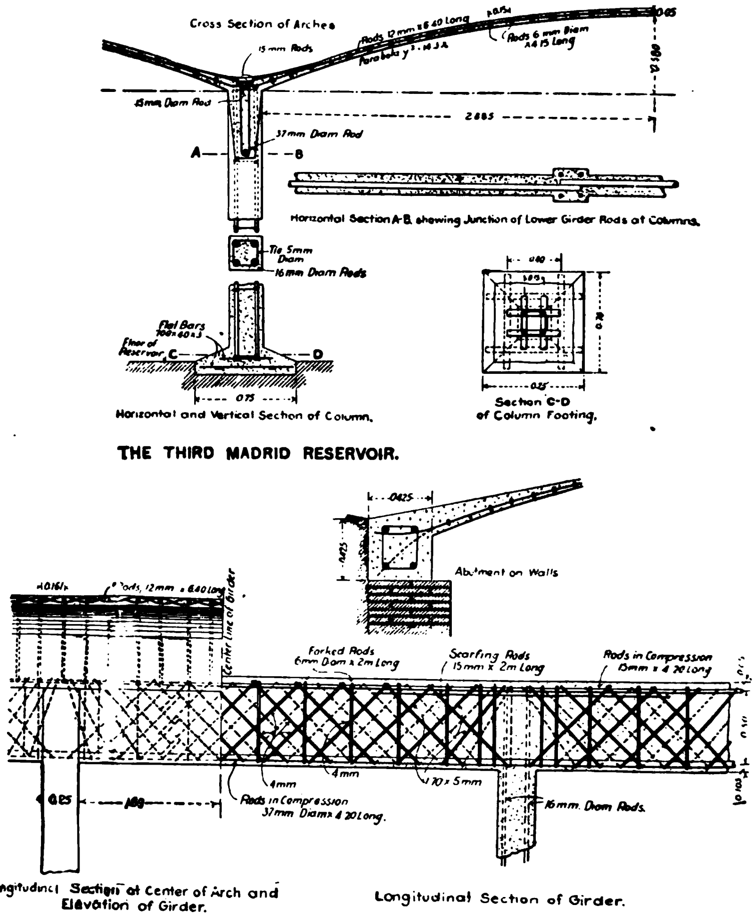


Fig. 522.—Roof Construction for Reservoir at Madrid, Spain.

the conditions affecting it when in the process of construction being entirely neglected. Then, too, the extremely slender section used and the lack of rigidity of the completed structure made its successful completion, at the best, a precarious piece of construction.

Grain Elevator Bins.—Reinforced concrete has been practically the only material used in Europe for many years in the construction of grain elevators. These bins are usually rectangular in shape and are supported on reinforced concrete columns. In this country both circular and rectangular bins have been built. Grain elevators are usually what are known as terminal elevators, viz., they are located at the terminal of a railway, grain being unloaded at such a point, cleaned, mixed and stored preparatory to being shipped by vessel to some distant point. This class of elevator requires a working floor story of 20 or 25 ft. between the bottom of the bins and the level of the railway tracks, two of which usually run through the house to permit the cars to be unloaded directly into the elevator bins. The machinery for unloading, cleaning and the mixing machinery are located on this floor. To secure the proper head room columns of upwards of 20 ft. in length must be used to support the girders carrying the bins proper, and must be so spaced as to allow the passage of the cars underneath. These requirements and others to secure the proper location of a line of cars so that a number of cars may be conveniently unloaded at one time fix closely the size of bin and the particular form of construction to be adopted.

The Canadian Pacific Grain Elevator, Port Arthur, Ont.—The Canadian Pacific elevator, which was completed in 1904, has a capacity of 443,000 bushels and consists of nine circular bins 30 ft. in diameter and 90 ft. high, and four inner bins formed by the walls of the circular bins. The centers of the bins are at the intersection of three longitudinal and three transverse lines 30 ft. apart in both directions. Fig. 523 is a part plan showing the arrangement of four circular bins and one inner bin of approximately rectangular form.

The walls are of reinforced concrete 9 ins. thick on foundations 24 ins. thick carried down to offset footings resting on hardpan. The conical tank bottoms are seated on rammed sand fill. Under the center of each row there is a concrete-lined conveyor tunnel 7 ft. wide, 7 ft. high and about 86 ft. long.

The walls of the cylindrical bins have a uniform thickness of 9 ins., except where the adjacent bins are tangent, and the acute angles between the convex surfaces are filled in solid with a width of 7 ft. and a maximum thickness of 2½ ft. The concrete used was composed of 1 part Portland cement, 3 parts sand and 5 parts Lake Superior gravel.

The horizontal reinforcement consists of hooping bars spaced 12 ins. apart vertically, the size of the bars decreasing from the bottom upwards. These bars are in pairs, one near each surface of the shell. For the first 15 ft. above the base their cross-section is 1 sq. in., for the next 31 ft. it is 0.88 sq. in., for the next

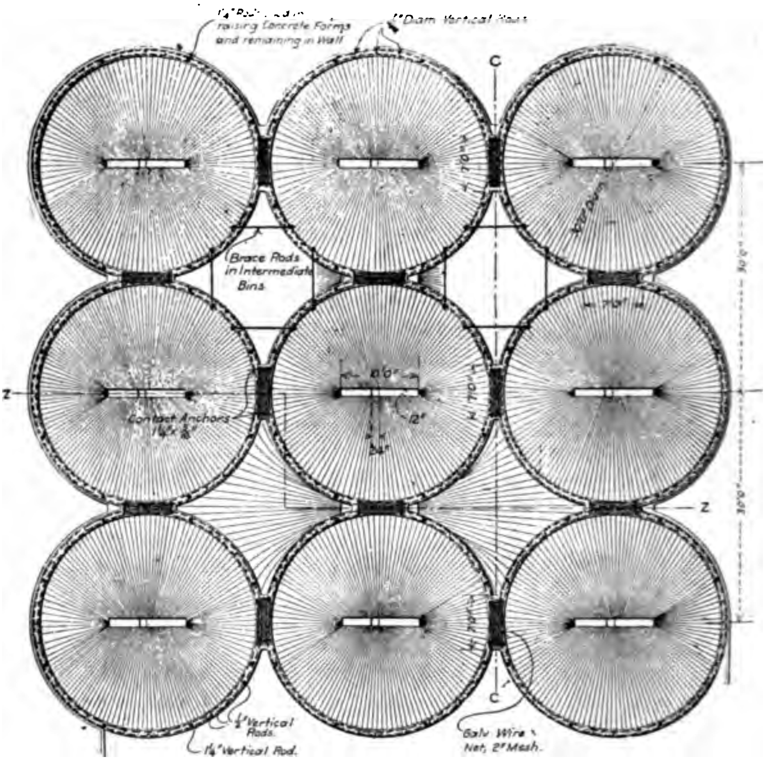


Fig. 523.—Part Plan of Grain Elevator at Port Arthur, Ont.

20 ft. it is 0.75 sq. in., and above that it is 0.50 sq. in. Besides the horizontal bars there are in each bin 27 vertical bars, ½ in. in diameter, spaced equally distant apart. Where the walls are thickened at the contact points of tangencies they are reinforced by horizontal layers of 2-in. mesh galvanized wire netting 12 ins. apart vertically. At these points they are also reinforced by two horizontal 1¼ × $\frac{5}{16}$ in. contact anchors 12 ins. apart vertically, which hook over the hoop bands.

The spaces enclosed between the convex surfaces of each group of four bins are also used for grain storage, forming quadrilateral bins with sides concave outwards. In other elevators, notably the elevators at Duluth, Minn., no provision was made for strengthening the convex circular walls of the enclosing bins against pressure when the circular bins were empty, with the result that they collapsed. A description of the Duluth failure is given below. To care for the unbalanced pressure, caused by the filling of these bins when the main circular bins are empty, tension rods were passed through the opposite walls near each corner, as shown on the partial plan (Fig. 523). These tie rods have a diameter of 1¾ ins. up to a height of 20 ft., 1⅝ ins. for the next 30 ft., 1½ ins. for the next 20 ft. and 1⅛ ins. from that point to the top. Their screw ends have nuts bearing on beveled washers, which are seated on flat steel tie-plates, which distribute the pressure over the cylindrical walls and connect the ends of adjacent rods. The bottoms of the circular bins are approximately conical surfaces, and consist of 3 ins. of concrete finished with 1½ ins. of Portland cement mortar. The concrete was laid directly on well rammed sand, and is without reinforcement. The sides deviate from a true cone, converging to a flat chute 10 ft. long and 1 ft. wide, with a 15 × 15-in. gate near the centers, through which the contents are discharged to the conveyor belt in the tunnel below. The rectangular inner bins have their bottoms highest in the center and slope to outlets at one corner of each, from which the grain is carried by chutes to the center of the tunnel over the conveyor belts.

The concrete walls of the bins were made in movable cylindrical forms 4 ft. high. The curved surface of the forms were made of 2-in. vertical planks, spiked to inside and outside circular horizontal ribs. The ribs were made like ordinary arch centers, with four thicknesses of 2 × 8-in. scarf planks bolted together to break joints and make complete circles inside the tank and circular segments of 270° or less on the outsides of the tanks. The moulds were faced on the inside with No. 28 galvanized steel and were maintained in concentric positions with a fixed distance between them by means of eight U-shaped steel yokes in radial planes. Each yoke consisted of an inside and outside vertical post with radial web and flanges engaging the inner and outer faces of the circular chords.

The posts project about 2 ft. above the tops of the moulds. and were rigidly connected there by means of heavy braces and an adjustable tension rod. These yokes were bolted to the inner and outer chords of the moulds and held them rigidly together. The lower ends of the vertical yoke posts were seated on jack screws, which were supported on falsework built up inside the tanks as the walls progressed.

The Failure of the Duluth, Minn., Grain Elevator.—The arrangement of the bins of the Duluth elevator resembles somewhat that of the Canadian Pacific elevator, described above. There was this difference, however: The sides of adjacent circular bins were not tangent, and did not have either the thickened buttresses between adjacent bins or tie rods to resist the thrust of the grain from within the inner rectangular bin.

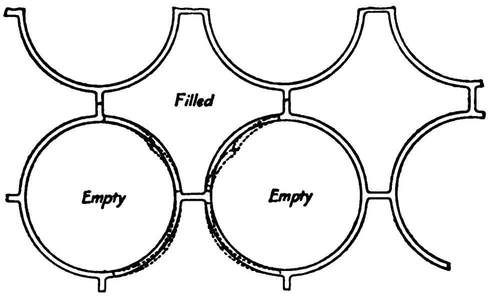


Fig. 524.—Diagram Showing Failure of Grain Elevator at Duluth Minn.

Figure 524 shows the arrangement of the enclosing walls. the entire absence of anything like a buttress, there only being a thin connecting wall, while in the Canadian Pacific elevator the walls are tangent and thickened. Failure took place by the crushing of the arched enclosure, there being no skewback to resist the thrust when pressure was brought upon the sides of the bin. The manner of failure is shown by the dotted lines in the figure.

The Duluth elevator consists of a series of circular bins about 33 ft. in diameter and 104 ft. in height. The first failure occurred on December 12, 1900, and a second failure occurred on April 16, 1903. The sides of the bins were 12 ins. in thickness at the bottom and 9 ins. at the top. The reinforcement consisted of 1½ × ⅜-in. steel bands spaced about 12 ins. centers. It is stated that the gravel concrete used for side walls was found to contain a high percentage of voids, and also showed evidences of containing a considerable percentage of foreign matter, as chips, bark, etc., which was gathered up with the gravel when the concrete was mixed. The lesson to be drawn from the above failures is that careful designing is necessary for success in this type of structure. No possible arrangement of loading should be overlooked while making the design, and, lastly, good materials should be used in fabricating the concrete.

Sand Storage Bins.—Two sand storage bins, having a capacity of 1,140 tons, or 850 cu. yds. each, of dry sand, were designed and constructed by the Turner Construction Co. for J. B. King & Co., at Hempstead Harbor, Long Island, in 1904. The combined weight of the bins and sand, which amounts to about 1,355 tons, is carried on fifteen columns. The tank, which is 27 ft. high and 30 ft. in diameter, is supported at a height of about 20 ft. above the pile foundation. The columns are arranged in two concentric arches, with a central column under the apex of the conical bottom of the bin. Each column has a square concrete footing 15 ins. thick and 4½ or 5½ ft. long on each side, depending upon whether it is designed for five or eight piles. The pile foundation consists of 96 12-in. piles about 25 ft. long. The column footings are connected by radial and circumferential beams of concrete reinforced with pairs of ¾-in. bars, as indicated on the plans, and the center group is connected by a horizontal concrete diaphragm 14 ins. thick. The footing under each column is reinforced by six longitudinal and six transverse ¾-in. bars 5 ft. 3 ins. long.

The center column and the six columns of the inner ring are each 22 ins. square, and are reinforced with four ¾-in. vertical bars, while the outer columns are 22 x 18 ins., with four ¾-in. vertical bars. The sections and details of tops of columns are shown in Fig. 525. The upper ends of all except the center columns engage the lower surface of two annular reinforced girders made by thickening the bottom of the bin, and are integral with it. Great pains were taken to make this conical surface and its girders monolithic, and the concrete for them was all placed in one day. In calculating the bottom and sides of the bins, fluid pressure for dry sand weighing 100 lbs. per cu. ft. was assumed.

The side walls of the bin are reinforced by horizontal circular rings, each ring being made of several rods, overlapping about

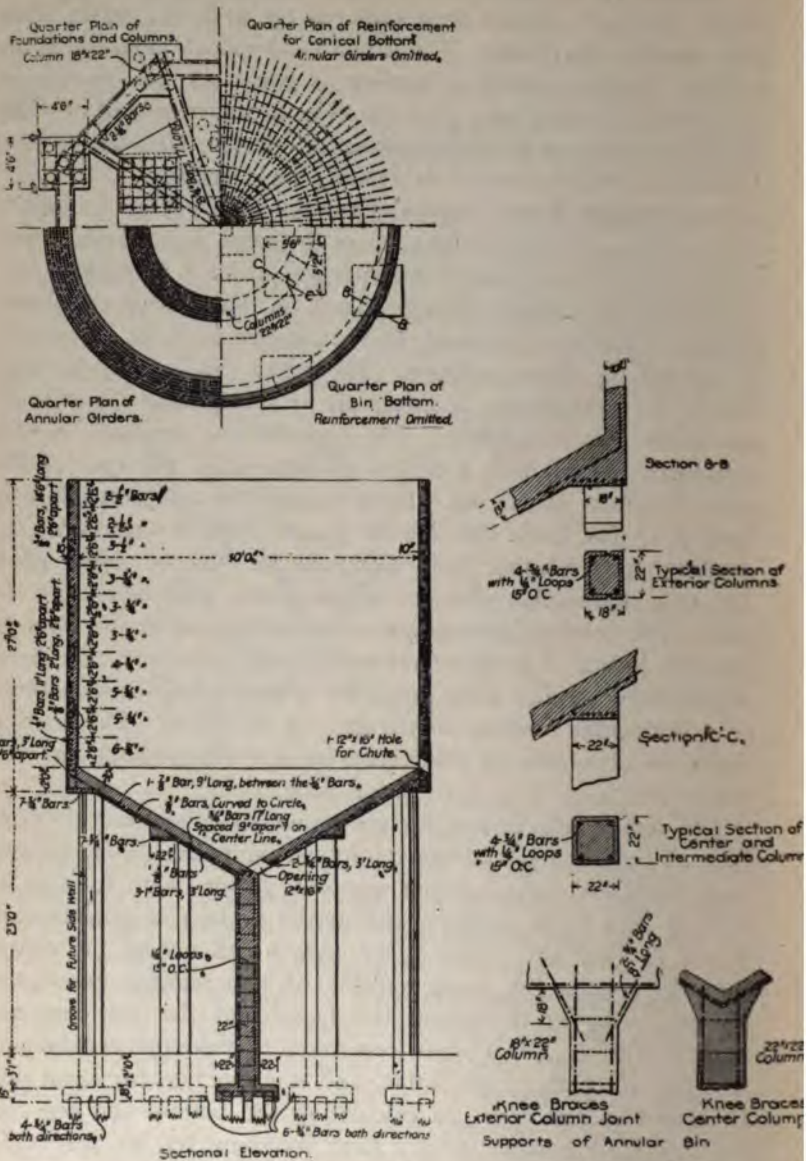


Fig. 525.—Details of Sand Storage Bins at Hempstead, Long Island.

24 ins. and wired together so as to be thoroughly spliced by the concrete. The sectional area of these rods is varied to conform with the different pressures assumed at different heights of the bin, by changing the number and size of the bars in 2½-ft. vertical zones of the wall, as shown in vertical section. The walls are also reinforced by two sets of vertical rods, the lower set consisting of larger bars than the upper set and overlapping them 6 ins. at a point a little below the center of the bin. The vertical walls of the bin were made in 5-ft. courses, which were additionally bonded together by ⅜-in. vertical dowels 2 ft. long, set in the center of the wall, 2½ ft. apart. The walls have a uniform thickness of 10 ins. in the vertical sides. The bottom is 13 ins. thick, and is reinforced by horizontal circular and inclined radial rods, as shown in drawing. The moulds for each bin were made with planed vertical staves 1⅛ ins. thick, forming panels 5 ft. high and about 8 ft. long. The staves were nailed to 2 x 10-in. horizontal segmental ribs, one at the upper edge, one at the lower edge and one at the center of each panel. The positions of these ribs were displaced 2 ins. vertically in adjacent panels, and they were extended beyond the edges of the panels so as to overlap each other and receive the splice bolts by which they were fastened together. The ribs were braced by inside and outside vertical standards 4 ft. apart on centers, each made with a pair of 2 x 6-in. strips 6 ft. long, blocked 1 in. apart and tied together by temporary radial bolts through the moulds. These bolts pass through sleeves of 16-gauge block sheet iron, bent cold, permanently bedded in the concrete and eventually filled with mortar. Enough moulds were made for a complete course about one tank.

After the concrete was about 24 hours old, the tie-bolts were removed from the moulds and they were lifted by a small hoist until the lower edges engaged the top of the concrete for about 2 ins. and the vertical standards engaged it about 1 ft. They were then supported on bolts through the upper part of the concrete. Each 5-ft. course was made monolithic by continuous concreting in one day's work. The conical hopper bottom was also built monolithic in one day's continuous concreting and was constructed without an inside mould.

All the work was done from outside platforms, supported at the level of the top of the forms by falsework built up from the ground level. No scaffold or platform was provided on the inside of the bin.

Coal Pocket for Pennsylvania Cement Co.—Figure 526 is a view of a coal pocket of simple design. The construction consists of transverse buttresses 9 ft. 6 ins. centers supporting a 6-in. slab reinforced with ½-in. corrugated rods spaced from 4 to 7 ins. centers. The 4-in. roof is reinforced with ½-in. transverse bars spaced 8 ins. and ½-in. longitudinal bars 2 ft. on centers. The roof is supported by longitudinal beams and struts reinforced in the usual manner. The roof is sloped so that it will not be

Fig. 526.—Coal Pockets, Pennsylvania Cement Co.

subjected to internal pressure. The roof house contains the conveying machinery for filling the coal pocket, while the tunnel beneath the pocket contains conveying machinery for removing the coal.

Atlantic City Coal Pocket.—Figure 527 shows details of a coal bin for the Water Department of Atlantic City, N. J., at Absecon Pumping Station. The total height from top of foundation to top of roof is 44 ft. 7½ ins.; the outside diameter is 36 ft. and inside diameter is 30 ft. A feature to be noted is the annular form of the foundation. The sides and bottom are 9 ins. thick. The bottom is reinforced with ½-in. annular bars, spaced 12 ins.

centers, and ¾-in. radial bars 18 ft. long, 2 ft. of which is in the side wall and 16 ft. in bottom and spaced 15 ins. centers and ¾-in. radial bars between the forms, also spaced 15 ins. centers, ¾-in. vertical bars, 22 ft. long, spaced 3 ft. centers, having the lower 3 ft. bent into the bottom, with ½-in. bars, 10 ft. long, to form the vertical reinforcement, while ⅜ and ¼-in. bars, spaced from 2 to 6 ins. centers, as shown, form the horizontal reinforcement of the side walls.

The roof is pyramidal in form, having a 10-in. 25-lb. I-beam

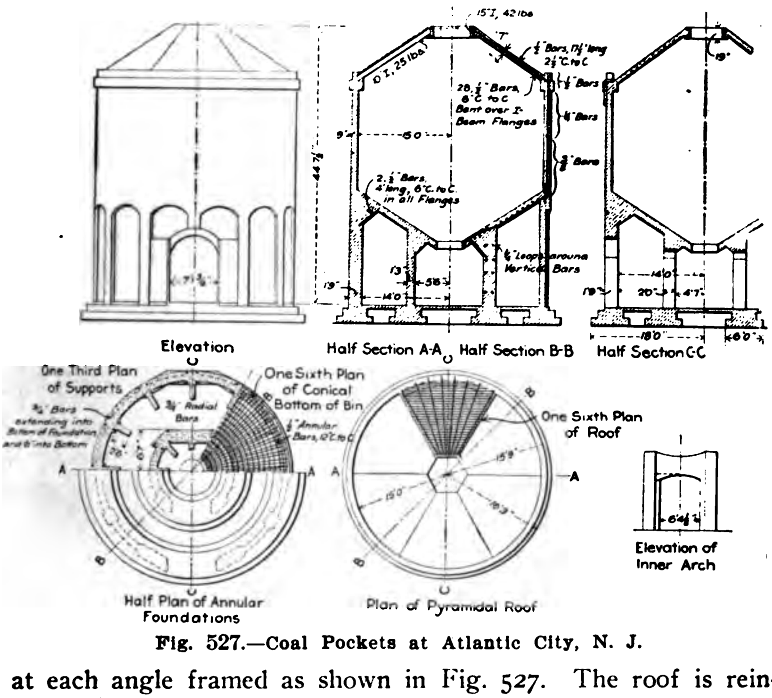


Fig. 527.—Coal Pockets at Atlantic City, N. J.

at each angle framed as shown in Fig. 527. The roof is reinforced with ½-in. bars bent over the I-beam flanges and spaced as shown. The general details of construction are shown on the drawings.

Concrete Gas Holder Tank.—Concrete has been used both in Europe and this country in the construction of gas holder tanks. The half section of tank, recently constructed by the Central Union Gas Company at the foot of 136th Street, near Locust Avenue, New York City, is shown in Figs. 528 and 529. This tank has an extreme diameter of 189 ft., and a depth of 41½ ft.

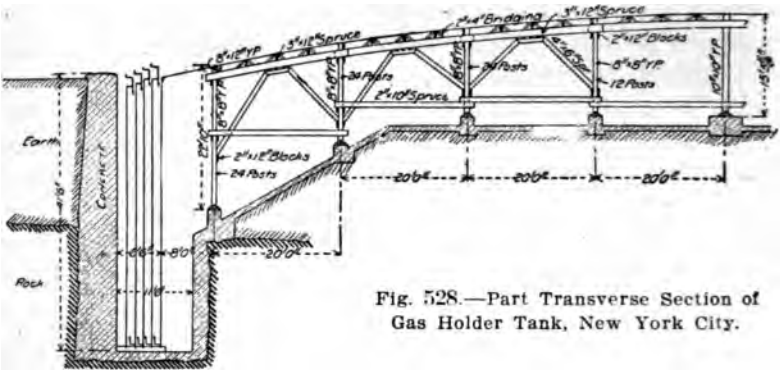


Fig. 528.—Part Transverse Section of Gas Holder Tank, New York City.

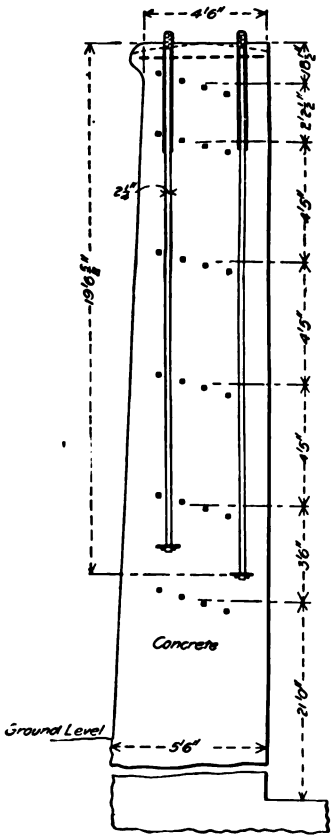


Fig. 529.—Wall for Gas Holder Tank at New York City.

Figure 529 shows a partial section of the monolithic concrete exterior wall, which is 42½ ft. high from the bottom of the footing and 5½ ft. thick at the base. Concentric with this wall is an inner one 166 ft. in external diameter and about 16½ ft. high above the footing. The top of this wall is continuous with a concrete lining 12 ins. thick, which forms the bottom of the tank, and is approximately a truncated cone. The annular space between the inner and outer walls serves to hold the water, which forms a seal for the bottom of the telescopic cylindrical steel gas holder shell.

The outer wall is reinforced in the upper part with six sets of horizontal circular bands of square twisted steel bars. Each bar of the upper and two lower sets has a sectional area of 1.25 sq. ins., while the intermediate sets have an area 0.75 sq. in. for each bar. In each set there are four complete rings or hoops, each made up of several pieces of twisted steel, with their ends lapping about 32 ins., and rigidly clamped together by U-bolts, with tie pieces screwed close against the bars. The splices in adjacent bars are made to break joints at least 2 ft. As shown in the figure, the hoops were not placed in the same horizontal plane.

The top of the wall is finished with a 6-in. coping reinforced by a continuous horizontal sheet of 3 in. No. 10 expanded metal. On top of the wall there are horizontal seats for the 20 vertical

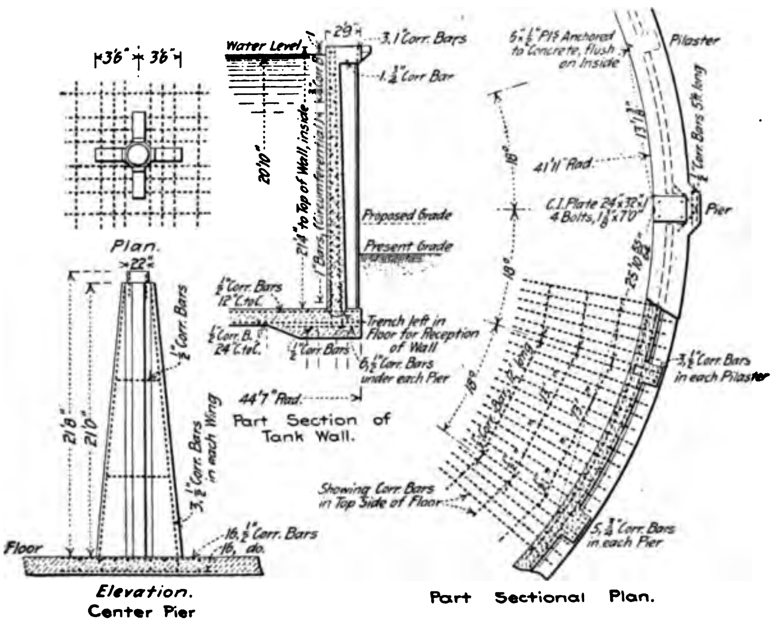


Fig. 530.—Details of Gas Holder Tank at Dubuque, Ia.

columns of the gas holder guide frame. At each seat are four vertical anchor bolts, 2¼ in. in diameter and 19 and 20 ft. long. These rods have at their lower ends forged heads engaging sockets in wrought iron ancl.or plates bedded in the concrete. The upper ends of the rods are in sleeves 4 ft. long and 4 ins. in diameter, so as to provide for a slight lateral movement and allow for adjustment to the front framework. Radial brick landing piers, with granite caps, are provided on the bottom of the tank, as shown in the figure. A 1:2:4 Portland cement concrete, with an upper finish of 1½ ins. of 1:2 cement mortar was used.

Gas Holder Tank, Dubuque, Ia.—The construction details of a gas holder tank, recently constructed for the Key City Gas Co., Dubuque, Ia., are shown in Fig. 530. This tank is 84 ft. in diameter and 21 ft. 5 ins. deep. The bottom of the tank is 5 ft. below the level of the ground, and rests upon a pile foundation. The floor is 16 ins. thick, and rests directly upon piles spaced 4 ft. 6 ins. centers. Under the walls the concrete is 28 ins. thick, and the pile foundation is reduced to 2 ft. 6 ins. Reinforced concrete tunnels are run under the floor for inlet and outlet pipes. The wall of the tank is 18 ins. thick at the bottom and 12 ins. at the top. A center pier, also of reinforced concrete, as shown, is also used. A 1 : 2½ : 5 concrete of limestone, all passing a 2-in. ring, was used for the floor, while a 1 : 2 : 4 mixture was used for the walls. The size and location of reinforcing rods are shown in Fig. 530.

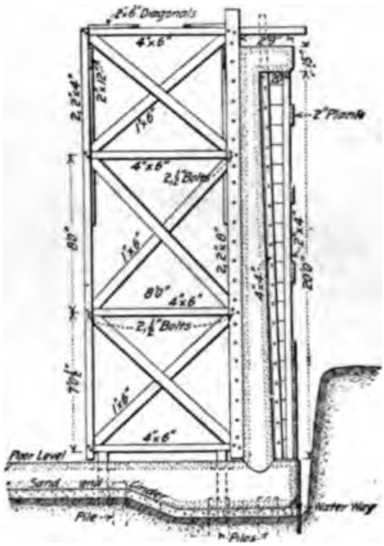


Fig. 531.—Method of Bracing Wall Forms, Dubuque Gas Holder Tank.

The method of bracing wall forms is shown in Fig. 531; while one panel of the inside forms is shown in Fig. 532. The outside forms were similar, but were concave instead of convex.

The circumferential bars for the walls were used in 30-ft. lengths, and spliced by lapping 3 ft.

Mr. John E Conzelman, Asst. Engr. of the St. Louis Expanded Metal and Corrugated Bar Co., who designed and built this structure, states in Engineering News, August 9, 1906, that

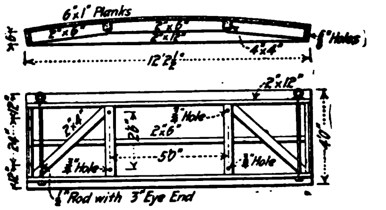


Fig. 2.—Wall Forms for Dubuque Gas Holder Tank.

the cost of unloading steel and placing it in structure was $7 per ton; that the labor of mixing and placing concrete amounted to 3.4 hrs. per cu. yd. for the floor, and 5.2 hrs. per cu. yd. for the wall (including pilasters and piers). The cost of forms averaged 9 cts. per sq. ft. of wall.

CHAPTER XXIX.

CHIMNEYS, TUNNELS, SUBWAYS, RAILROAD TIES, FENCE POSTS, PIERS AND WHARVES.

Chimneys.—For high chimney construction reinforced concrete is not only superior to brick or steel as regards stability and strength, but it possesses great durability with practically no maintenance charges. Many chimneys have been constructed of this material in the past few years, and the popularity of this type of construction is increasing. Examples of a number of structures will be given, together with a description of the general methods of construction.

The reinforced concrete chimney for the forge shop of the United Shoe Machinery Co., at Beverley, Mass., is a good example of a chimney of rectangular section, adapted to industrial works, when a chimney of moderate height is needed. This chimney is 77 ft. 10 ins. in height and tapers from 9 ft. 3 ins. x 10 ft. at the base to 7 x 6 ft. at the top. It is divided into two portions by an interior concrete diaphragm, reinforced by horizontal ¼-in. bars spaced 18 ins. apart up to 25 ft. above the footing. One side of the chimney is again subdivided by a longitudinal diaphragm, making two flues, one of which is used for furnaces and the other for an induced draught. The upper 45 ft. of the chimney is unlined, while the portion below is lined with firebrick. The walls have a thickness varying from 18 ins. at the base to 9 ins. at the top, and are reinforced by six vertical ½-in. bars lapped 18 ins. at the joints and spliced to make them continuous from the base to the top; ¼-in. horizontal bars bent to form rectangular frames connect the vertical bars. These horizontal bars are placed 1 in. from the outside of the concrete, and 12 ins. apart vertically throughout the full height of the chimney. The foundation is a mass of concrete 6 ft. 6 ins. deep, and is stepped off to form a base 13 x 14 ft. Figure 533 shows the general features of the chimney.

Most notable among the early chimneys built of reinforced concrete is that for the Pacific Electric Ry., Los Angeles, Cal. This chimney is of Ransome construction, and was designed by Mr. Carl Leonardt, of Los Angeles, and merits description, as it possesses a number of features not commonly met with in this type of construction.

The Los Angeles chimney is 180 ft. in height above its base,

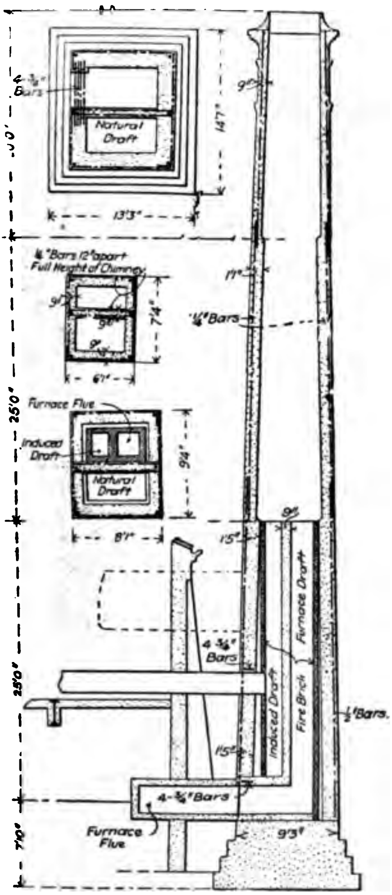


Fig. 533.—Rectangular Chimney, United Shoe Machinery Co.'s Forge Shop, Beverly, Mass.

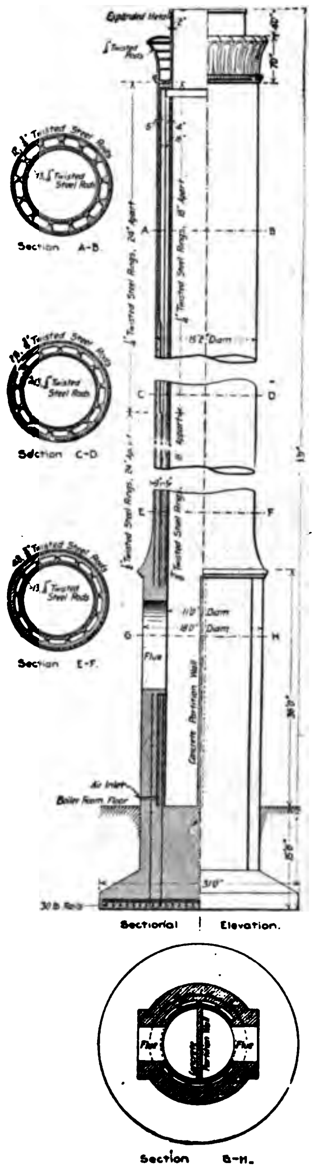


Fig. 534.—Chimney for Power House, Pacific Electric Ry., Los Angeles, Cal.

which is 15 ft. 6 ins. below the level of the ground. This chimney is rectangular in section to a height of 36 ft. above the grade, at which point it assumes a circular form, with an exterior diameter of 15 ft. 2 ins., the inner diameter being about 11 ft. The rectangular form for the lower portion of the chimney was necessitated by the entrance of two flues from opposite sides. The construction of the chimney is further shown in the accom-

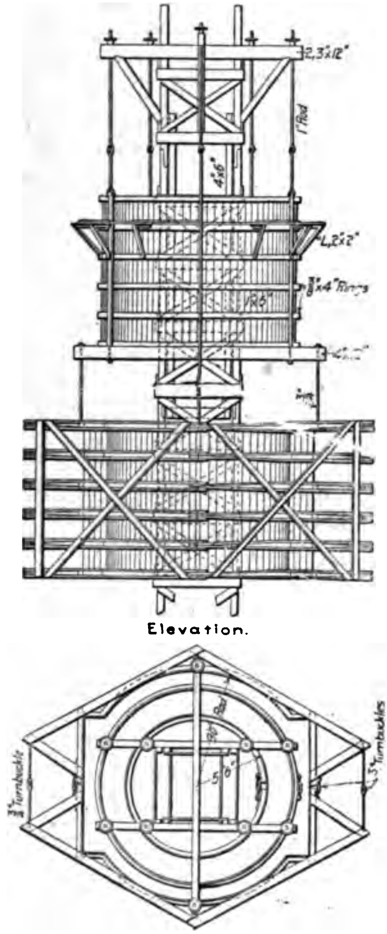


Fig. 535.—Tower and Falsework for Building Los Angeles Chimney.

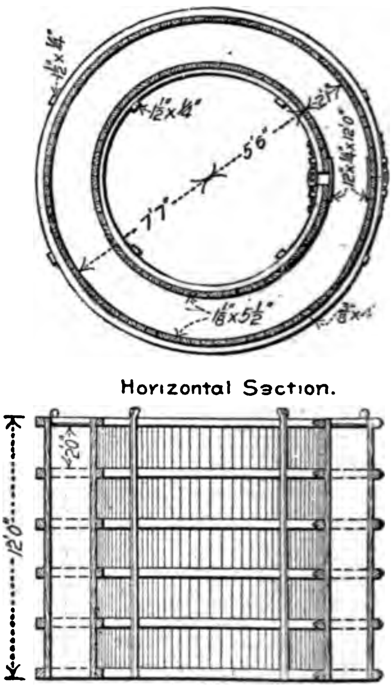
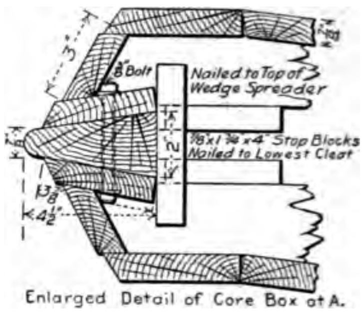


Fig. 536.—Moulds for Shells, Los Angeles Chimney.

panying figures. It consists of two concentric walls independent of each other from base to top, and separated by an air space of 11 ins. to 16 ins., increasing in width toward the top. The outer shell above the rectangular part is 9 ins., 6 ins. and 5 ins., respectively, up to the cap in sections of about equal height; while the inner shell is 5 ins., 4½ ins. and 4 ins. thick, respectively, from bottom to top in corresponding sections. The inner shell ends 4 ft. below the cap, and is free to elongate by heat independently of the outer shell. As shown in the sections, Fig. 534, it will be seen that at intervals of 30 ins., measured around the chimney, the air space is contracted for a length of 6 ins., and reduced to the width of 2½ ins.; at every 5 ft. in height this is again reduced to ¾ in. by the introduction of a concrete brick in the wall. In this manner the oscillations of either shell independent of the other is checked, and the outer shell may sway in the wind ¾ in. without bringing pressure upon the inner shell.

Ransome square cold twisted steel bars were used for the vertical and horizontal reinforcements in each shell. The horizontal reinforcements consist of ¼-in. bars placed at intervals averaging 18 ins. in the inner shell and 24 ins. in the outer; ¾-in. vertical bars were placed 1 ft. apart in the lower one-third of the shell above the flues, 2 ft. apart in the middle and 4 ft. apart at the top section of the outer shell. In the inner shell ¼-in. bars were used, spaced about 3 ft. apart in the circumference of the shell. The concrete for the outer shell consisted of 1 part Portland cement and 2 parts sand and 6 parts crushed granite, but that for the inner shell consisted of 1 part Portland cement, 2 parts sand and 4 parts broken sandstone.

In the construction of this chimney a temporary tower of timber (Fig. 535) was erected inside the chimney, its top being kept well above the highest level of the concrete, and from this tower the moulds are hung by adjustable suspender rods. All material was hoisted up the shaft by an electric hoist. The tower scaffolding consisted of four 4 × 6-in. timbers having uprights with 2 × 10-in. horizontal bars bolted thereto every 5 ft., and 1 × 6-in. cross-pieces. The head or top scaffolding was formed of 6 × 14-in. timbers, to which hoisting rods were attached. To avoid the labor of dismantling this head scaffolding for each set of moulds, telescope scaffolding was placed inside the main upright scaffolding, which enabled the workmen to disconnect the head scaffolding, raise the entire head intact, and put in extensions to the uprights, all of which could be done in about 2½ hours. The cross beams at the top of the tower supporting the moulds consisted of two pairs of beams about 16 ft. long, each cantilevered about 5 ft. beyond the sides of the tower. The inner and outer moulds were each suspended from these cross beams by four equally spaced vertical rods having threaded tops, which engaged screw wheels bearing on the beams. A light working platform projected out from the outside mould near the top, and the concrete platform was hung from this mould a little below to catch dropping material or a fallen workman. Inside the chimney a staging was supported from the tower, on which the workmen stood in placing and tamping the concrete. The



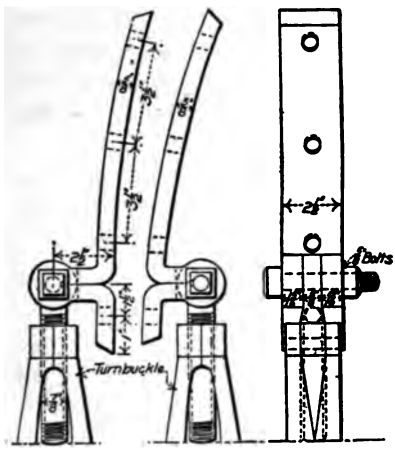


Fig. 537.—Joints for Hoops for Shell Moulds, Los Angeles Chimney.

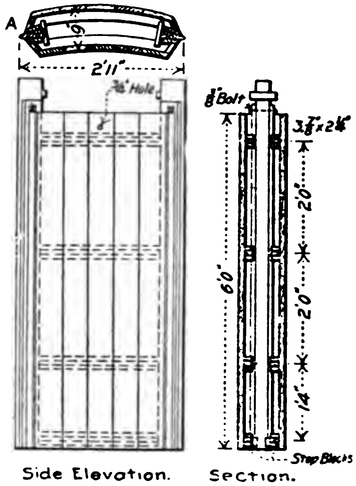


Fig. 538.—Core Boxes Used with Shell Moulds, Los Angeles, Cal.

shell moulds are shown in Fig. 536. These were made with vertical wooden staves 12 ft. long beveled to an angle of 10 degrees on both edges, so as to be in contact on the face next the concrete, and having a V-shaped opening on the opposite face. The staves were locked together with bands built up of ⅜-in. strips of Oregon fir to a thickness of 5 ins. and a width of 4 ins. The ends of these bands were connected by specially designed

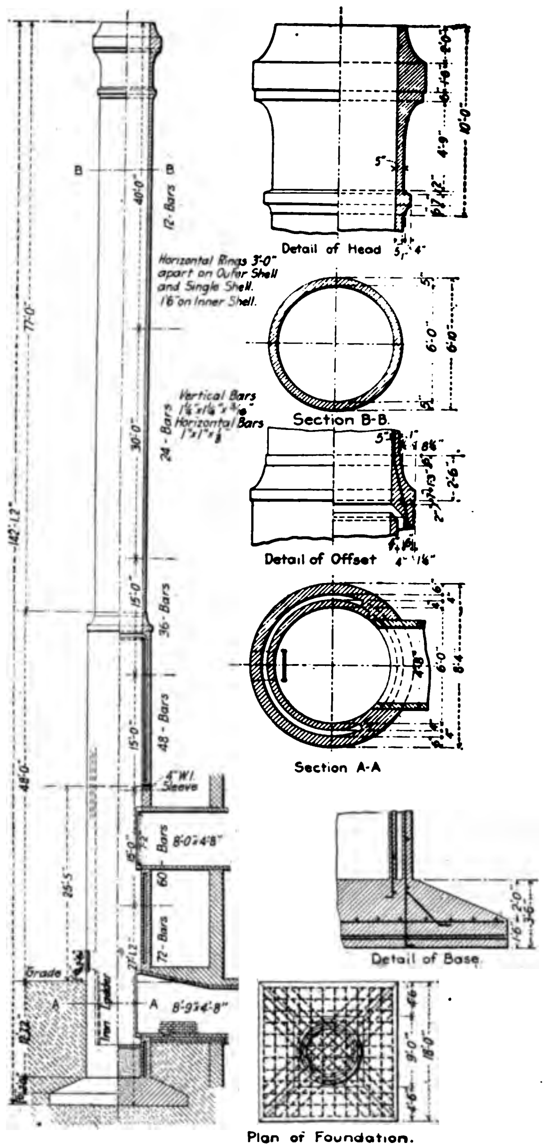


Fig. 539.—Chimney for United Shoe Machinery Co.'s Works, Beverly, Mass.

jaw-forgings and sleeve nuts (Fig. 537). Six hoops were used for each mould, being located outside the stave in the outside mould and inside the staves for the inside moulds. To attach the moulds to the supporting beams $1\frac{1}{2} \times \frac{1}{4}$-in. bars, with an eye at the top and a length of 12 ft., were bolted to each hoop in a vertical position at the four points of the circumference directly between the suspender bars. A detail of the core-boxes for forming the spaces between the inner and outer shells is shown in Fig. 538. The concreting was done in five sections, one section being completed every day.

Probably more reinforced concrete chimneys have been con structed by the Weber Steel Concrete Chimney Co., of Chicago, than by any other concern in the world. The author is indebted to the company for the following information. The features claimed for this chimney are possessed to a greater or less extent by all reinforced concrete chimneys, and may with much advantage be enumerated in this place.

In the construction of these chimneys the work is carried on continuously from the foundation to the top, thereby forming a monolith. The chimney is airtight, and this, with the smooth inner shell, gives a high working capacity. The construction of reinforced concrete chimneys proceeds with greater rapidity than brick chimneys. Another feature is their light weight, they being lighter than brick, and hence requiring smaller foundations. A concrete chimney resists the influence of chimney gas and of heating better than one made of other materials. The use of concrete gives an opportunity for improving the appearance of the chimney without excessive cost for ornamentation.

Figure 539 shows the details of a chimney of moderate height built for the United Shoe Machinery Company, Beverly, Mass., while Fig. 540 is a view of the chimney completed. This chimney is 142 ft. 1 in. in height from the bottom of the foundation to the top, and 6 ft. in diameter. The foundation extends about 16 ft. below ground. The shell is double to the height of 48 ft. above ground. The inner shell is 4 ins. thick, while the outer one is 6 ins., and the upper portion of the chimney shell is 5 ins. in thickness. The reinforcing bars consist of $1\frac{1}{4} \times 1\frac{1}{4} \times \frac{3}{16}$ ins. vertical and $1 \times 1 \times \frac{1}{8}$-in. horizontal T-bars. The number of bars in the circumference of the chimney shell and the method of arranging the bars in the base, together with the details of construction, are shown in Fig. 539. The horizontal rings are spaced 1 ft. 6 ins. centers in the inner shell and 3 ft. centers in the outer and upper single shells.

Another example of Weber chimney (Fig. 541) is the one used for the Butte Reduction Works, at Butte, Montana, to carry off the gases and fumes from the copper smelting furnaces. This chimney is the largest reinforced concrete chimney that has thus far been built. It stands on a base of slag 12½ ft. high, making its top 352½ ft. above the surface grade. The shell has an inner diameter of 10 ft. This chimney was designed to resist

Fig. 540.—View of United Shoe Machinery Co.'s Chimney.

the pressure of the wind blowing at a velocity of 100 miles an hour. The base is 42½ ft. square, 8 ft. thick, and made of 1 : 3 : 5 concrete reinforced as shown in the drawings. The shell is double to a height of 101 ft., and above this point the single shell rises 231 ft., making a total height above the slag base of 340 ft. The outer shell is 9 ins. thick, and the inner one 5 ins., with an air space of 4 ins. between. The single upper shell is 7 ins. thick. A 1 : 4 concrete is used for all the shells. The reinforcing bars in the base and for the verticals are 1¼ × 1¼ × 3/16-in. T-bars, while the horizontal rings are 1 × 1 × ⅛-in. T-bars. A working stress of 16,000 lbs. per sq. in. was used for the steel. The rein-

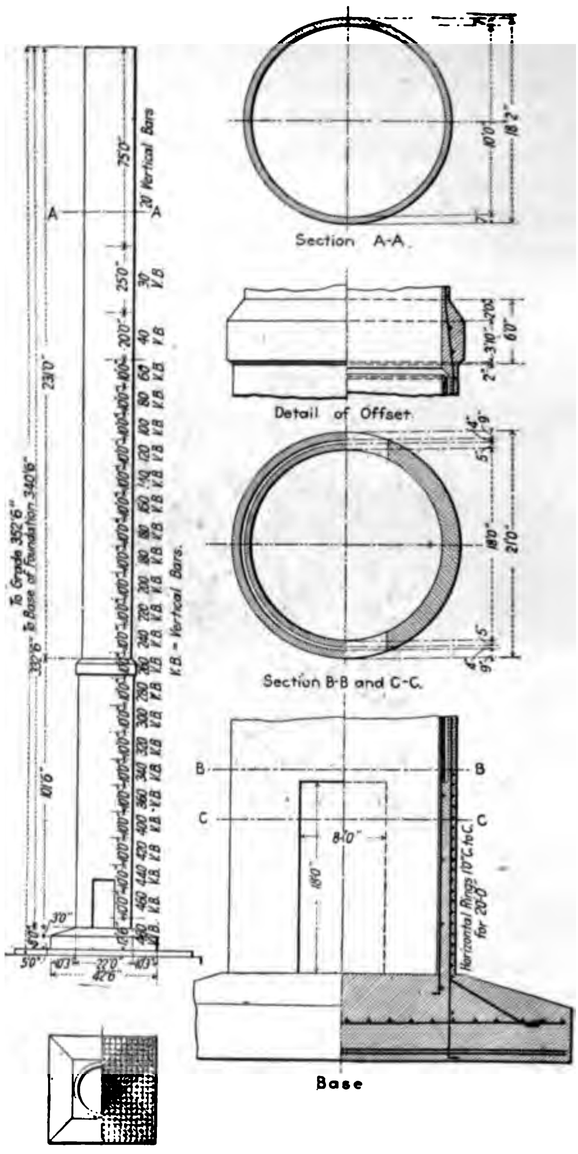


Fig. 541.—Chimney for the Butte Reduction Works, Butte, Mont.

forcement in the base consists of two layers of 20 bars each, crossing at right angles, and two layers of 13 bars each, running diagonally. In the outer shell and the single shell, which is above, the reinforcement consists of 460 vertical bars at the bottom, decreasing gradually to 20 at the top. The inner shell has 20 vertical bars throughout its entire height. The chimney is so designed that no wind strain comes on the inner shell. The horizontal reinforcing rings are spaced 3 ft. apart in the outer and single shells and 18 ins. apart in the inner shell.

In the construction of Weber chimneys the forms consist of

Fig. 542.—View Showing Forms for Weber Chimney.

two rings of six sections, each about 3 ft. wide, fastened together with patented iron fastenings. The moulds are held in place by friction on the concrete only, and are disconnected before hauling up to the position required for the next course. The flat top ring shown in Fig. 542 is a patented guide ring to hold the vertical steel rods in alignment through holes in it. The ring is made of two ¾-in. layers of wood, and is pushed on ahead of the centers. It also carries the beam for the hoisting pulley. All materials are hoisted inside of the chimney, no interior scaffolding whatever being needed. For the double shell one form or ring a day is filled, while for the single shell two forms a day are usually filled. The T-section reinforcements are spliced by lapping about 2 ft. The character of the forms is shown in Figs. 542 and 543.

Tunnels.—Concrete, both plain and reinforced, has been extensively used for tunnel and subway construction during the past few years. The principles of design and methods of con-

Fig. 543.—View Showing Forms for Weber Chimney.

struction are similar to those for sewer and water conduits. The larger sections used and general conditions met with in this class of work make the stresses to be dealt with large and the construction correspondingly more difficult.

The term tunnel is usually applied to construction under cover, in which the tunnel bore is advanced by drifting, the surface of the ground above not being disturbed. On the other hand, a subway is usually distinguished from a tunnel as being a construction in open cut. The tunnel usually consists of a single arch spanning the opening, while to save head room the roof of a subway is usually flat, being supported by roof beams or girders, sometimes carried at an intermediate point or points by columns.

The New York Subway.—The original section for the New York Subway consists of steel bents spaced 5 ft. on centers, having jack arches of concrete sprung between the beams to form the roof and side walls.

The bents consisted of a roof beam 26 ft. long for a double track section, supported by two side and one intermediate column. The foundation of the columns and floor of the sub-

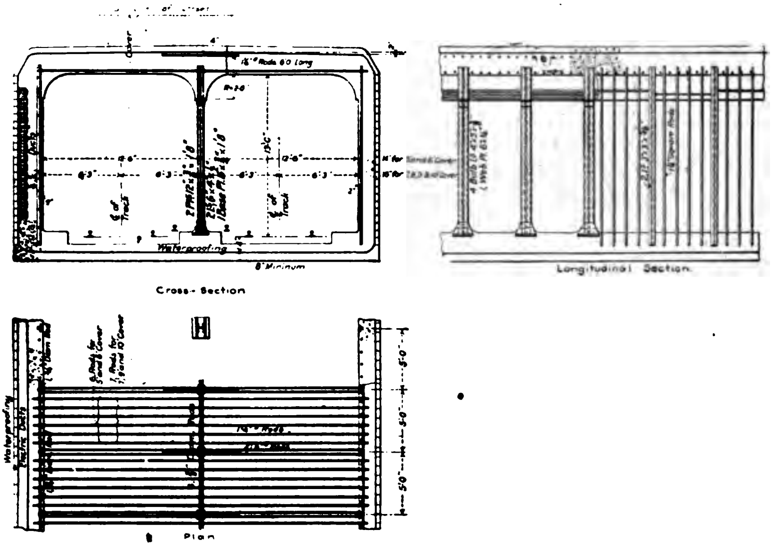


Fig. 544.—Subway Construction New York Rapid Transit R.R.

way also consists of concrete. The size of roof beam is governed by the height of cover, a 15-in. 60-lb. beam being used for ordinary conditions where the cover varies from 5 to 10 ft. The side column consists of a 12-in. 40-lb. I-beam, while the center column is made up of four angles of special section and one plate.

In the construction of Contract No. 2, known as the Brooklyn Extension, the standard section is reinforced concrete for the roof and side walls and a steel column similar to that used in the original section to support the roof slab at mid span. The thickness of roof slab and amount of reinforcement varies with the cover. For a 5-ft. cover the slab is 18½ ins. thick, and is reinforced with 1¼-in. square rods spaced about 8-in. centers. Two 1⅛-in. diameter rods connect the center column to a two-angle post or wall reinforcement spaced also 5-ft. centers. Three ⅝-in. diameter rods connect the center columns together at the top.

The side walls for a 5-ft. cover have a thickness of 14 ins.

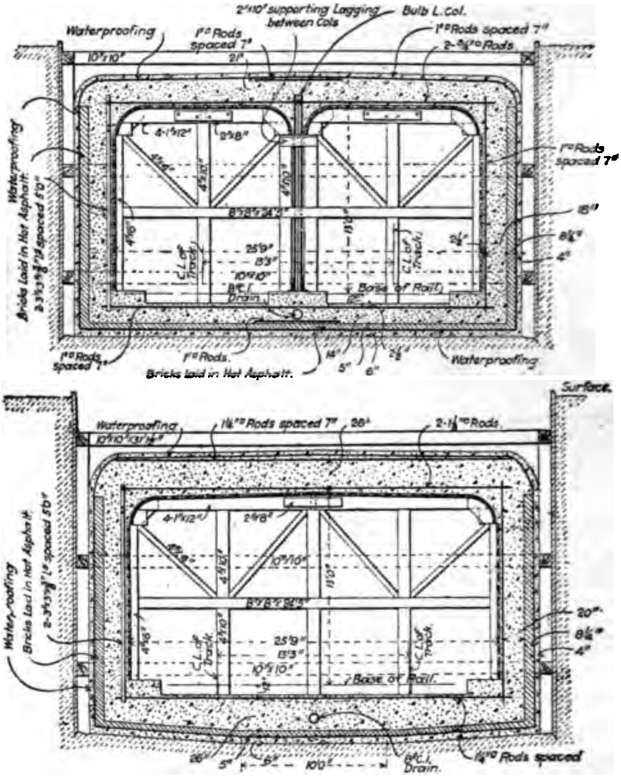


Fig. 545.—Forms for Reinforced Concrete Subway, New York Rapid Transit R.R.

and have square vertical rods between angle uprights spaced about 1-ft. centers. The arrangement of the reinforcement, together with general features, will be understood by consulting Fig. 544. The arrangement of the centering used in the construction of a section at Battery Park loop is shown in Fig. 545.

The Philadelphia Subway.—In the construction of the Philadelphia subway the roof beams and interior columns similar to

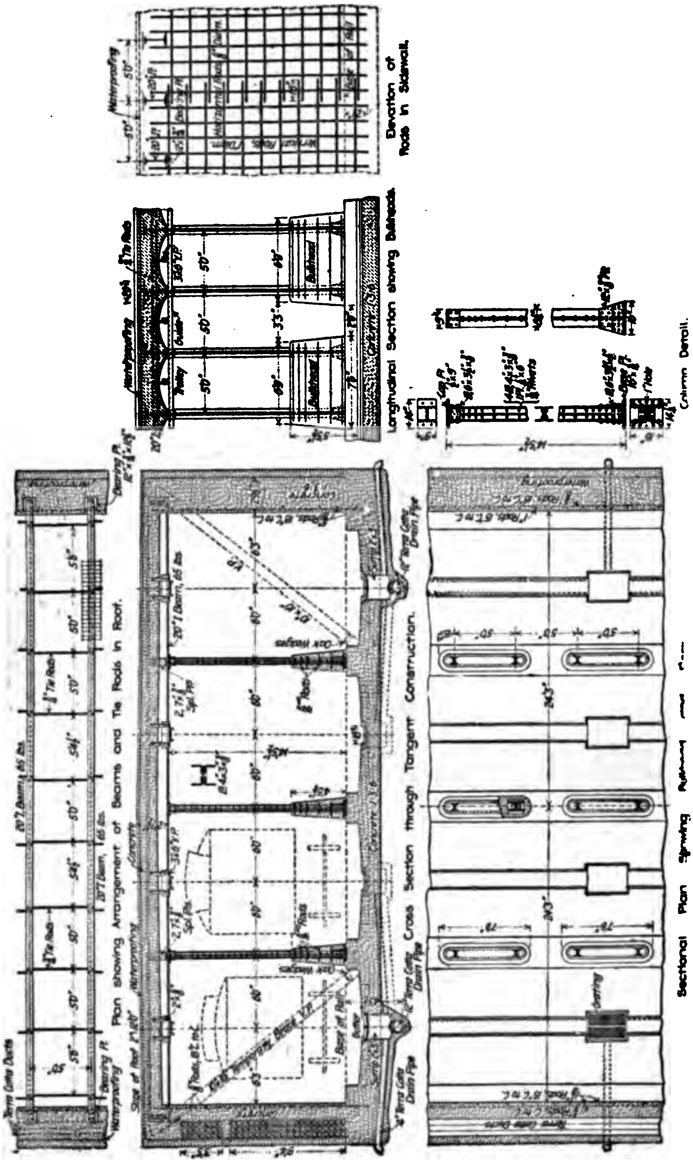


Fig. 546.—Details of Philadelphia Rapid Transit Railway Subway.

those used for the original New York subway section are retained The side walls are, however, reinforced with 1-in. diameter vertical round rods, spaced about 1 ft. centers and horizontal rods ⅝-in. diameter spaced about 16 ins. centers. A bulkhead of concrete reinforced with longitudinal rods about 5½ ft. high joined the posts in pairs. This bulkhead is intended to prevent the knocking out of the interior posts in case of a derailment. These details are all shown by Fig. 546.

Aspen Tunnel, Union Pacific R. R.—In the construction of the Aspen Tunnel, on the Union Pacific R. R., at certain points, unusual pressures occurred on account of the unstable nature of the

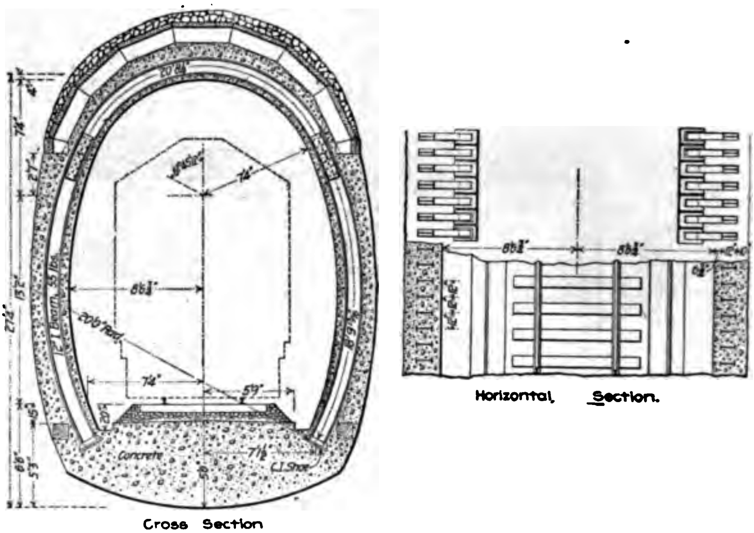


Fig. 547.—Aspen Tunnel, Union Pacific Ry.

rock. The tunnel lining used consisted of concrete stiffened with steel ribs spaced from 12 to 24 ins. centers. These ribs consist of 12-in. 55-lb. I-beams, in three sections, bent to the required curve and riveted together. The ribs are embedded in the concrete, which reaches from 4 ins. to 6¾ ins. inside the ribs and extend backwards to the wall of the excavation. The construction is shown by Fig. 547. The invert is a mass of concrete reinforced with old rails placed transversely to the axis of the tunnel.

The East Boston Tunnel.—This tunnel was constructed with the aid of compressed air. The lining is of concrete, 2 ft. 9 ins.

thick at the side and crown, while the invert is 2 ft. thick, and with the exception of a twisted bar at the crown is unreinforced. The centering was a steel framework, as shown in Fig. 548, for a part of the work; the framework for the remainder was of wood.

In the construction of this tunnel two side drifts 8 ft. square were first driven a certain distance and solidly timbered. The bottom of the drifts were then excavated, and the side foundations of concrete were placed in lengths of from 16 to 20 ft. After the foundations had set the interior forms for the side walls were placed upon them, and the concrete side walls, 3 ft.

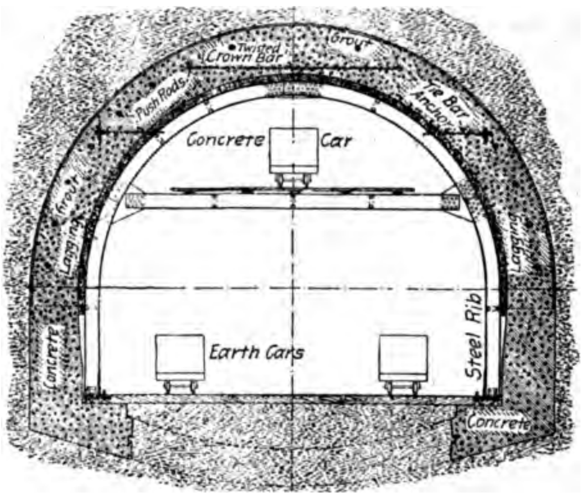


Fig. 548.—Section of East Boston Tunnel, Showing Reinforcement and Forms.

thick, built up to within 16 ins. of the springing line of the arch. This work was kept upwards of 100 ft. in advance of the shield. The shield moved upon rollers traveling upon these side walls as a track.

The main excavation was made under the shield, and the concrete placed in sections 2½ ft. long under the tail end of the shield. Sixteen cast-iron rods, 3 ins. in diameter and 2½ ft. long, were placed in the concrete the entire length of the tunnel, and the shield was pushed forward by means of hydraulic jacks pushing against the cast-iron rod. A 4-in. lagging was placed over the rib-centering. The centering consisted of steel ribs spaced 2½ ft. on centers. A 1 cement and 2 sand Portland cement grout was forced in on top of the arch to form a water-

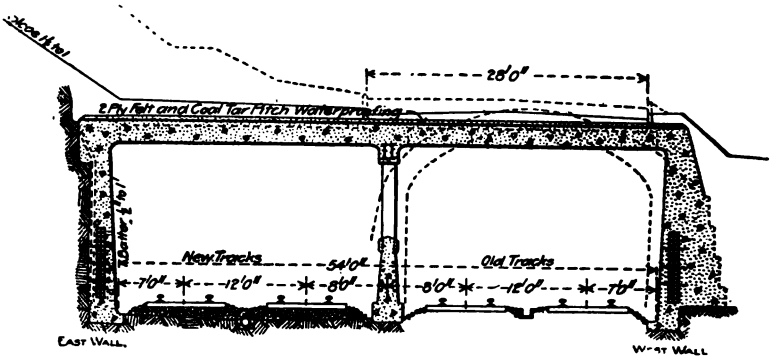


Fig. 549.—Cross-Section of Ossining Tunnel, N. Y. C. & H. R. R. R.

proofing film about 1¼ ins. thick. The invert was laid as the shield moved forward.

The Ossining Tunnel.—The improvements comprised in the

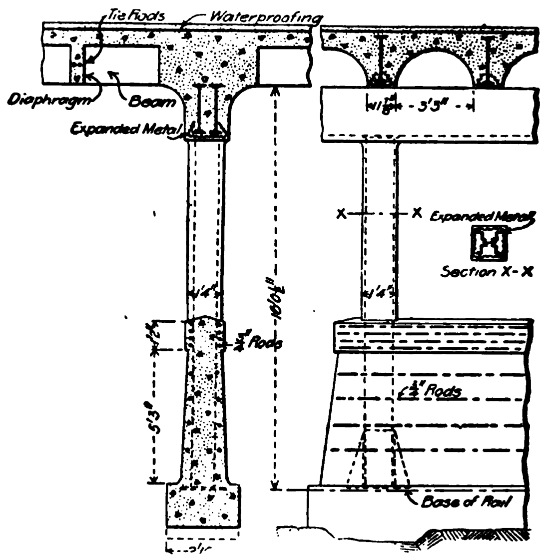


Fig. 550.—Part Longitudinal Section of Ossining Tunnel.

electrification of the metropolitan zone of the New York Central Railroad necessitated a change from a double track tunnel to a four-track tunnel at Ossining, New York. The tunnel is constructed on the bank of the river, the ground dipping sharply to the water's edge. The rock is of a micaceous variety, with an irregular stratification, dipping approximately at an angle of 45°.

A portion of the tracks at this point is in open cut, and a portion in a tunnel. When in open cut the sides are lined with concrete, and when necessary the concrete lining becomes a retaining wall.

Figures 549 and 550 show a section of the tunnel, together with details of construction. The columns, which are of a built-up, Z-bar section, surrounded by expanded metal and concrete, are spaced about 12 ft. 6 ins. centers on the longitudinal axis of the tunnel, and are arranged in groups of 4 or 5, with their bases united by a solid collision wall of concrete reinforced with horizontal rods to form piers intended to deflect the cars and

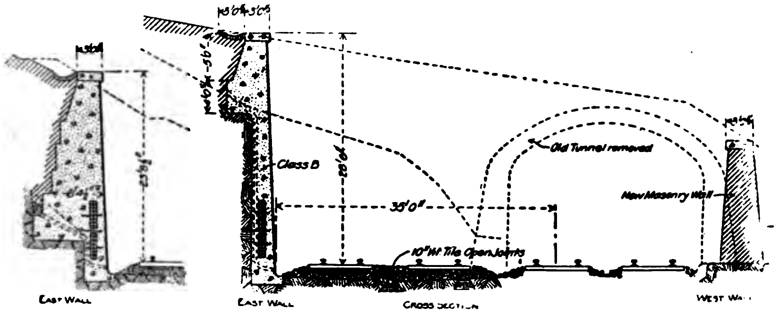


Fig. 551.—Section of Ossining Improvement, Showing Retaining Wall Work.

diminish impact in case of a derailment. The tops of the columns are connected by two longitudinal 24-in. I-beams encased in concrete. These beams support 20 and 24-in. I-beams spaced about 4 ft. on centers. Jack arches of concrete are sprung between these transverse beams forming the roof of the tunnel. The concrete has a minimum thickness of 6 ins. This arch construction is stiffened by a series of ribs about 12 ins. wide the full depth of the cover, and spaced about 4 ft. 6 ins. on centers.

The roof and walls are waterproofed with a coating of tar about ¼ in. thick.

Figure 551 shows a section of the concrete retaining wall used for an open cut section of this work. A section of the old tunnel which was removed is shown in dotted lines.

Railway Ties.—Railway engineers have been experimenting for years to find a substitute for wooden cross ties which will possess all the good points of the wood tie, and, while cheap in first cost, will be more permanent. Reinforced concrete ties have been tested to a limited extent on short stretches of track, and, while the experimental sections are not long enough and have not been tested freely enough as yet to warrant final conclusions, the success of the concrete tie has, however, been great enough to promise that ultimately a satisfactory design will be secured.

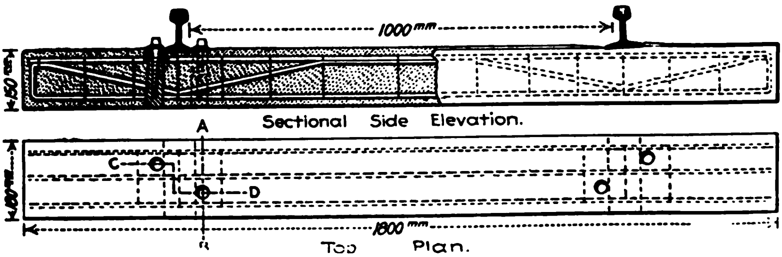


Fig. 552.—Cross-Tie Used on a French Railway.

Figures 552 and 553 show details of a cross tie used on a French railway of one metre gauge, from Vairon to Saint Bèron. Sixty cross ties were tested for a year, and proved so successful that 250 more were ordered put in to continue the test on a larger scale. These ties are about 7 × 5½ ins. × 6 ft. The reinforcement consists of three trusslike frames, each formed of a

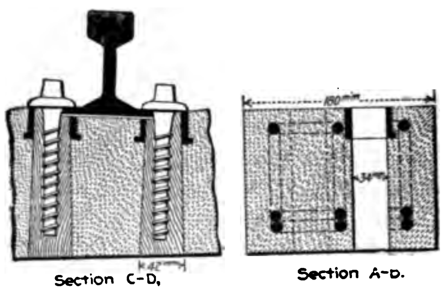


Fig. 553.—Rail Fastening Used with Tie Shown by Fig. 552.

single rod, as shown in Fig. 552. The upper and lower parts of each frame are tied together and the three frames are tied together across the tie. A depression is provided for the rail seat and wedge-shaped holes are moulded through the body of the tie for wooden plugs, in which screw bolts for the rail fastenings are driven (Fig. 553). Wooden tie plates are placed under the rail when it is set. These ties weigh about 230 pounds and cost in the neighborhood of 90 cents.

Ulster and Delaware R. R. Tie.—Fig. 554 shows details of a tie which has been successfully tested on the Ulster and Delaware Railroad. The reinforcement consists of a steel angle with both legs turned down; holes properly spaced are punched through the

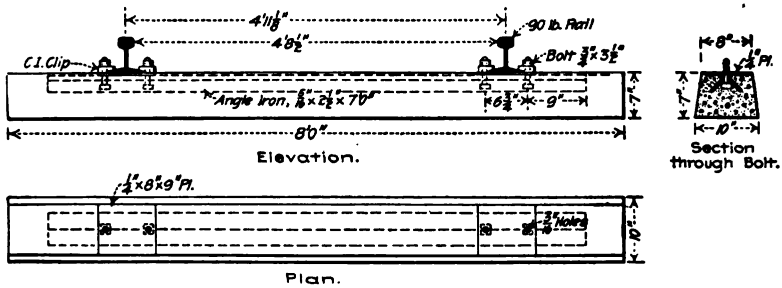


Fig. 554.—Cross-Tie, Ulster & Delaware R. R.

angle for the bolts which form the rail fastening. A square headed bolt ¾ × 3½ ins. is used and the two legs of the angle hold it firmly in place and keep it from turning when the nuts are tightened.

An iron tie plate is used. The nuts of the bolts, contrary to expectation, did not work loose, but after 18 months of continuous

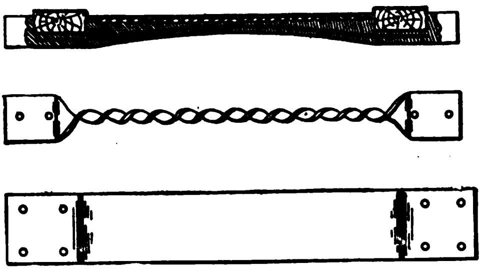


Fig. 555.—Cross-Tie, Hecla Belt Line Ry., Hecla, Mich.

use were as tight as the day the tie was put in, and, moreover, no attention had been given to them since the ties were installed. These ties were made of a 1 : 2 : 4 Portland cement concrete and cost 42 cents to manufacture exclusive of the metal. The weight of the tie is about 450 lbs.

The principal objection to this tie appears to be the impossibility of renewing or replacing a bolt in case one is injured.

Hecla Belt Railway Tie.—A reinforced concrete tie tested on the Hecla Belt Line R. R., in Bay City, Mich., is shown in Fig. 555. The reinforcement consists of a lower flat bar and a twisted upper bar with flat ends, which extend outside of the concrete resting upon and holding in place wooden spiking blocks partly embedded in the concrete underneath the rails. Holes are punched in the top plate for spiking.

The Kimball R. R. Tie.—A cross tie of rather elaborate design is shown in Fig. 556. This tie is being tested on the Pere Marquette R. R. It consists of two rectangular blocks of concrete, each 7 x 9 ins. in cross section and 3 ft. long, reinforced and connected by two steel channels placed 2 ins. back to back. Hard

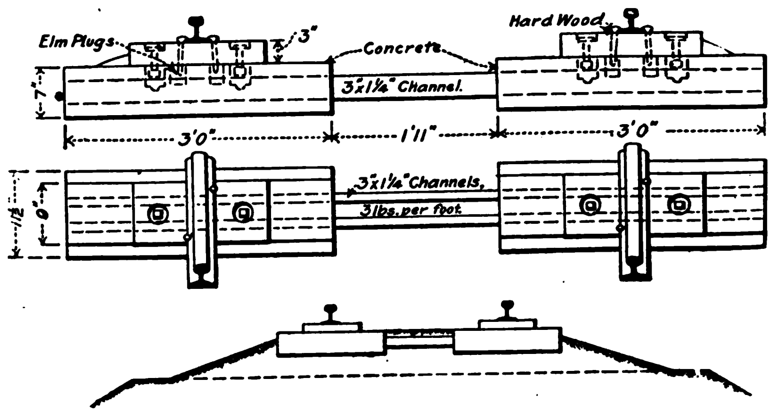


Fig. 556.—Cross-Tie, Pere Marquette R. R.

wood blocks 3 x 9 ins. x 1 ft. 6 ins. bolted to the concrete serve as spiking pieces. Cast-iron sockets moulded in the concrete hold the bolts for fastening the spiking pieces and space the channels. Wooden plugs set in holes moulded in the concrete receive the ends of the spikes. Exposed portions of the channels are covered with neat cement grout.

Buhrer Tie.—Figure 557 shows a concrete and steel cross-tie, of which upwards of 3,000 are in use on the Lake Shore and Michigan Southern Railway. As will be seen, this tie consists of a piece of an old 65-lb. rail turned upside down and imbedded in the concrete. A portion of the bottom is shaped not unlike an ordinary wooden tie. The flange of the old rail forms the seat for the track rail and to it are attached the rail fastenings. These ties weigh about 400 lbs. It is stated that under ordinary conditions this tie has proved very satisfactory.

In designing a tie provision should be made for the renewal of the fastenings in case they are injured in any way. The simpler the details and construction the greater will be the chance of success.

The great weight of the concrete ties, while making them diffi-

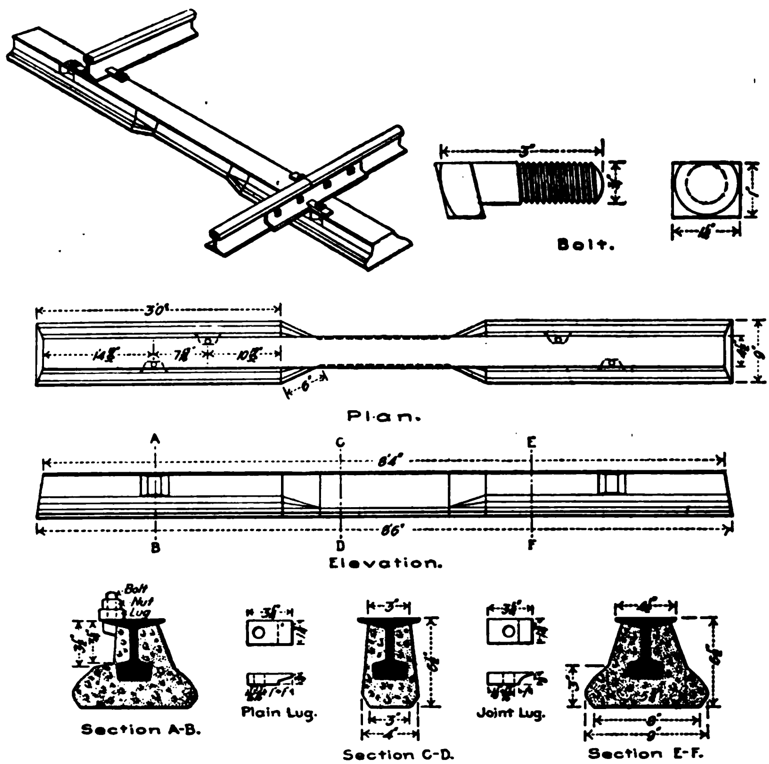


Fig. 557.—Cross-Tie, Lake Shore & Michigan Southern Ry.

cult to handle and surface, will when they are once in the tracks contribute greatly to its rigidity and permanence and will undoubtely reduce the maintenance charges for surfacing and lining track.

Reinforced Concrete Fence Posts.—Fence posts may be constructed of reinforced concrete and in many situations will in the long run prove less costly than wooden posts. The reinforced concrete post will not be affected by the weather and, hence, will last longer than either wood or iron. A concrete post may be consructed in advance and put in place after it has hardened and set sufficiently hard as not to be injured by handling. Posts may also be moulded in place, but the length of time which they must stand before the removal of the forms requires the use of a large number of forms, making this method of manufacture expensive. For the sake of ecenomy the post is usually tapered. It is customary to reinforce them with wire or light rods, one rod being placed near each corner. To provide a means for fastening the fence wires the simplest and most satisfactory method is to use large staples

Fig. 558.—View Showing Reinforced Concrete Posts in Fence.

having their ends bent so as to hook firmly into the concrete. These are put in their proper positions when the concrete is placed in the moulds.

Figure 558 shows a braced corner post and line of reinforced concrete posts moulded in place. Figure 559 shows view of mould for corner post and braces and Fig. 560 detail of mould for corner post and brace. These forms are patented by the Stiner Cement Fence Post Co., of Indianapolis, Ind. The moulds for the line posts are similar in construction (Fig. 561). The corner post here shown is 10 × 10 ins. at the top, 12 × 12 ins. at the ground, 5 ft. high, and extends about 3½ ft. into the ground. The hole in

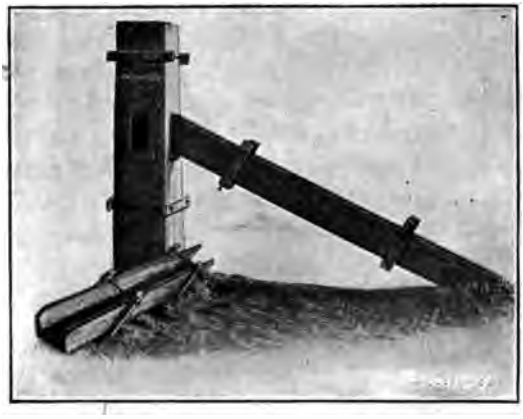


Fig. 559.—View of Mould for Corner Post.

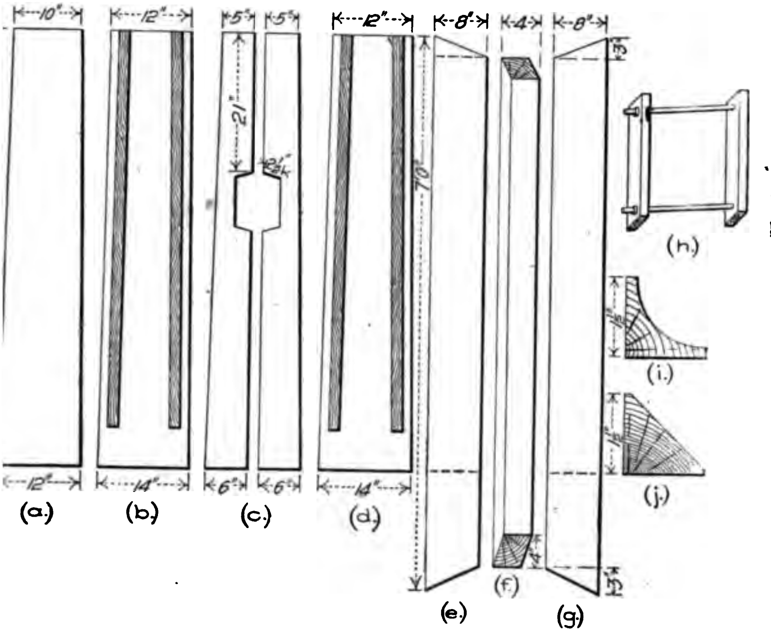


Fig. 560.—Details of Mould for Corner Post.

the ground may be flared outward as much as desired, giving a massive base, thereby increasing the stability of the post. In Fig. 560, a, b, c and d are the sides. The moulding i or j is nailed to the side pieces b and e as shown by the shaded portion. A hole is cut in part c for the connection of the brace. Parts e and g are the sides of the brace mould and part of the bottom, and part h shows clamps for holding together the mould. The line posts are 5 x 5 ins. at the top, 7 x 7 ins. at the ground and 5 ft. high. The moulds are quite simple, being two pieces 5 ft. 6 ins. long, 9 ins. at the bottom and 7 ins. at the top and two pieces 7 ins. at bottom 3 ins. at top, also 5 ft. 6 ins. long, all 1-in lumber. Two clamps (h) are used to hold them together.

Fig. 561.—Mould for Line Post.

After digging the post holes the forms are set up, the reinforcing wire or rods put in place. The concrete is then mixed and tamped into place. Staples may be put in position through holes cut in one side of the mould.

Figures 562 and 563 show two forms of moulds, one for posts with two sides beveled and the other with four sides beveled. With the moulds here shown the moulding is done horizontally. The mould consists of two end pieces having notches to hold in place the longitudinal boards, cross pieces are provided as shown to prevent bulging of the longitudinal pieces. Hooks may be used to hold together the various pieces forming the mould. The post for which mould shown in Fig. 562 is provided is 6 x 6 ins. at the base and 3 × 6 ins. at the top and 7 ft. long. If it is desired to chamfer the edges triangular strips of wood may be nailed at the edges of the mould.

The mould is placed on a platform and greased or coated with

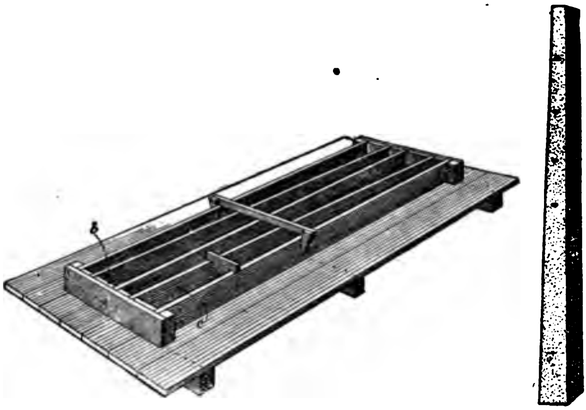
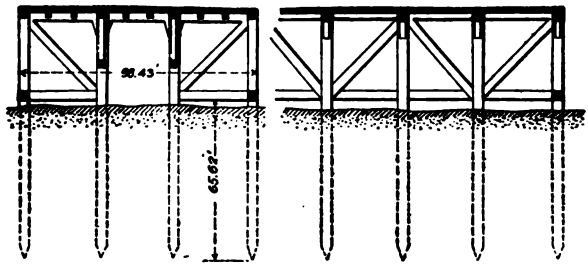


Fig. 562.—Mould for Fence Posts, with Two Tapering Sides.

soft soap. About 1 inch of concrete is deposited and carefully tamped in place, the rods put in place and concrete filled in until about 1 in. from the top yet remains; the top rods are then put in and the concrete finished off. The ends and sides of the mould

Fig. 563.—Mould for Fence Posts, with Four Tapering Sides.

may be removed in about 24 hours, but the posts should not be handled for about a week, during which time they should be sprinkled several times daily and protected from the sun and wind. The intermediate strips may be carefully withdrawn at the end of two or three days. If possible the posts should not be set until



Transverse Section. Longitudinal Section.

Fig. 564.—Sections of Ocean Pier at Southampton, England.

they are about 2 months old. A 1 : 2½ : 5 stone or gravel concrete with the stone or gravel under ½ in. has proved satisfactory for this class of work. Mr. Philip L. Wormeley, Jr., Farmers' Bulletin 235, U. S. Dept. of Agriculture, states that a post measuring 6 x 6 ins. at the bottom, 6 x 3 ins. at the top, 7 ft. long, should not cost more than 30 cents, the cost items being as follows: One cubic yard of concrete will make 20 posts of the size given above and for a 1 : 2½ : 5 mixture—

1.16 barrels of cement, at $2	$2.32
0.44 cubic yards of sand, at 75 cts.	.33
0.88 cu. yds. of gravel, at 75 cts.	.66
Cost of materials for 20 posts = 1 cu. yd. of concrete..	$3.31
Cost of concrete for 1 post	$0.17
Cost of 28 ft. of 0.16-in. steel wire, at 3 cts. per pound....	.06
Total cost	$0.23
Cost of mixing concrete, moulding and handling will not exceed	.07
Total cost of 1 post	$0.30

Of course, the costs of the materials and labor in any given locality will vary and affect the cost accordingly.

Piers and Docks.—Reinforced concrete has been used for piers, wharfs and docks in Europe, but until recently has been little employed in this country. The usual type of construction consisted of reinforced concrete piles braced by longitudinal and transverse struts, carrying beams and girders, supporting the floor slab. The size of the reinforcement used for the various members is essentially the same as that employed for similar members in other classes of construction.

Christophe describes a wharf of Hennebique construction at Woolston, Southampton, England. This structure is L-shaped, having the stems 90 × 31 and 100 × 47 ft., respectively. It was designed to carry a moving load of 500 lbs. per sq. ft. with a crane to lift 35 tons at the outer end. Figure 564 shows a cross and longitudinal section. The piles were spaced 10 ft. on centers. The details of the reinforcement of the piles and girders are shown in Fig. 565, together with connections of horizontal and diagonal bracing.

The most notable piece of pier construction thus far done in the United States is the reconstruction of the Atlantic City Steel Pier at Atlantic City, N. J. This work consists of strengthening the

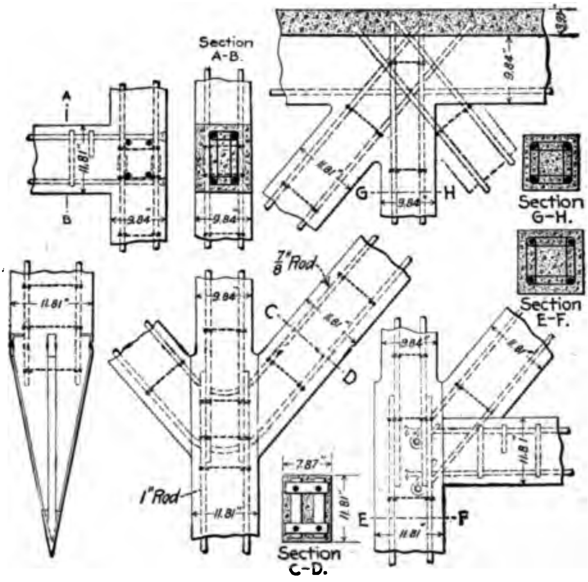


Fig. 565.—Details of Southampton Pier.

old iron pier, which had become so weakened by rust that it was necessary to either strengthen or rebuild it as a whole, and of adding side bents to increase the width of the pier.

The old cross and longitudinal girders were encased in a concrete beam 13 ins. wide by 27 ins. deep, reinforced at the top and bottom with 1-in. bars, as shown in Fig. 566.

The old steel pipe piers, which were 10¾ ins. outside diameter, were encased with a reinforced concrete shell built in place about the pile and sunk as the work progressed. A water jet was used to sink the shells until they rested upon the old cast-iron disks supporting the steel pile. The details of these pile reinforcements are shown in Fig. 566.

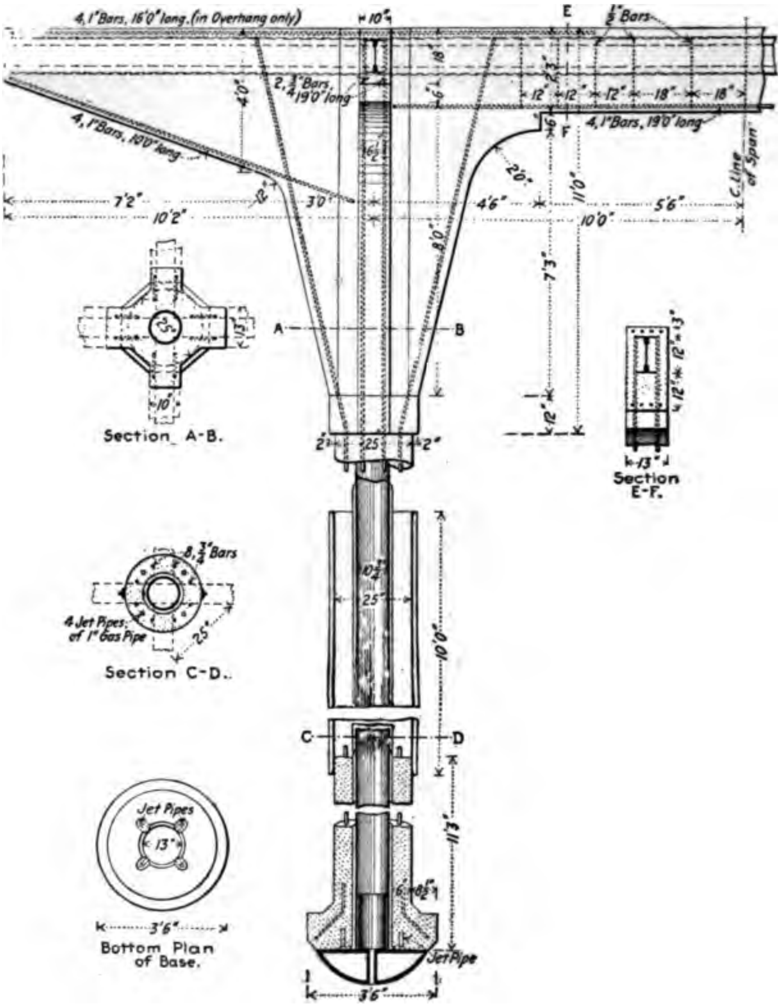


Fig. 566.—Section Showing Method of Encasing Steelwork of Atlantic City Pier with Concrete.

For the new work two sizes of piles were used, 12 ins. and 25 ins. in diameter, respectively. The transverse girders are 2 ft. 4 ins. deep and 7 ins. wide, while the longitudinal struts are 15 ft.

deep. Details of the girders, bracket and knee brace construction are shown by Fig. 567.

As will be seen, the lower end of the pile has a diameter of 2 ft. 6 ins. and together with the pile was built about a 2 in. jet pipe and reinforced with six ¾-in. bars. These piles were constructed in advance and sunk with a water jet having a pressure of 65 lbs. per sq. in. When in position the girder moulds were built in place and the girder reinforcement and concrete forming the girders and struts placed. The piles were sunk from 8 to 14 ft. into the

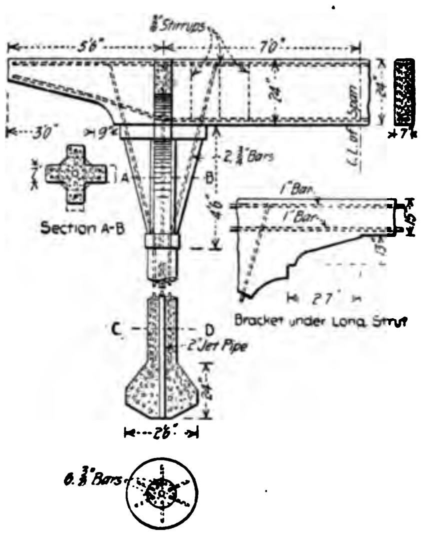


Fig. 567.—Details of 12-in. Reinforced Concrete Pile, Atlantic City Steel Pier.

sand and had a maximum length of 32 ft. 6 ins. The bearing power of the sand was taken at 5 tons per. sq. ft.

The 25-in. piles were carried down to a depth of 16 ft. into the sand. The details are similar to those of the 12-in. piles. The bottom 12 ft. of the 25-in. piles, including the bulb point, was moulded in wooden forms with the jet pipe and reinforcing rods in place. When the concrete had hardened a $\frac{3}{16}$-in. galvanized steel shell was slipped a short distance over the top of the concrete and the joint made watertight by calking with oakum. The steel shell was constructed watertight and was long enough to reach above the water when the pile was sunk into position. The reinforcing rods were then put in position and hooked to the lower sections and the pile and casing swung into place and jetted down with 85 lbs. water pressure. After being sunk the casing was filled with concrete and the bracketed tops, girders and struts moulded in place. The length of these 25-in. piles was 52 ft. from the top of the girders. A 1: 2: 4 Vulcanite Portland cement concrete was used. The concrete was mixed wet and puddled into the pile forms by means of bamboo fishing poles, which proved very satisfactory in giving a good mortar surface and working the mixture between the reinforcing rods and the face of the former. This construction was designed by the Concrete Steel Engineering Co. of New York.

CHAPTER XXX.

CONCRETE IN BRIDGE CONSTRUCTION.

Concrete, both reinforced and unreinforced, is extensively used in bridge construction. Without reinforcement it is used in the construction of arches up to quite wide spans; it is also used for abutments, piers, spandrel walls, etc. Reinforced concrete has a still broader use, being employed for both girder and arch bridges, spandrel walls, posts, floor slabs and beams, foundations, and for strengthening existing structures.

For convenience reinforced concrete bridges may be classified as girder and arch bridges. Girder bridges may consist of a reinforced slab with or without stiffening ribs. Without stiffening ribs girder bridges are only adapted to short spans, usually not exceeding about 15 ft. For wider spans the ribbed slab is employed, consisting of two or more heavily reinforced concrete girders connected by a reinforced slab, the construction being not unlike that used for wide span floor construction. Bridges of this type have been constructed with spans up to about 100 ft. Another type of girder bridge occasionally met with consists of a deep girder having portions of the web removed, the arrangement of the reinforcement being such as to bring about some form of truss action. Girder bridges, especially for the wider spans, are best adapted to the construction of foot bridges and highway bridges. When the span exceeds about 50 ft. the arch bridge will prove the more satisfactory form of construction. For railroad bridges, on account of the rough usage to which they are subjected, girder bridges are seldom employed for spans exceeding about 25 ft.

Arch bridges may consist of a solid or ribbed arch. The solid arch consists of a curved slab, which when reinforced has some one of the arrangements of reinforcement shown in Figs. 145 to 154, pages 257-9. Ribbed arches consist of two or more curved girders connected either by a thin reinforced slab or a framework of beams and girders. The reinforcement for ribbed arches consists usually of top and bottom reinforcing bars with or without connecting web members. Sometimes the skeleton consists of a rigidly connected built up truss. The slab or arch rib may be of uniform thickness throughout its whole length, but often is of varying thickness, increasing usually from the crown to the springing points. Sometimes, especially when hinges are used, the arch ring increases in thickness from the crown to the haunches and then decreases again to the springing points.

Either solid or skeleton spandrel construction may be used with both the solid and ribbed type of arch bridge. Skeleton spandrel construction, on account of its reduced weight and cost, is, however, most commonly used for the ribbed arch bridge and for the wider spans is becoming more popular for the solid arch. The skeleton spandrel construction may consist of either a series of spandrel arches carrying the roadway or of a framework of posts and girders carrying a reinforced slab forming the roadway. Both types of construction will be illustrated. Skeleton post and girder construction greatly reduce the dead weight to be carried and transmits the loads directly to the arch ring, thereby greatly simplifying the analysis of strains.

Culvert construction is essentially the same as that used for short span bridges, the only material difference being in the addition at times of wing walls to protect the embankment. A culvert may be considered as a short span bridge serving for a waterway through an earth embankment of greater or less height.

Floors of steel bridges may often be constructed with economy of reinforced concrete. Such floors replace the various types of metal trough floors used heretofore for highway and railroad bridges, and have in many respects proved much more satisfactory than metal floors.

Reinforced concrete may at times be used for strengthening old steel bridges, thereby lengthening their life and enabling increased loads to be carried without the great expense of a new structure.

If properly designed and constructed both concrete and reinforced concrete bridges are practically indestructible and, hence, possess great ultimate economy. Reinforced concrete bridges are considerably lighter than masonry or concrete bridges and do not bring so great weight upon the foundations, often giving a substantial saving in the cost of the latter. Again, on account of their great stiffness, due to the presence of the metal, reinforced concrete bridges possess greater security against danger caused by any slight settlement of the foundations. The steel provides resisting power against dangerous tensile strains due to any cause and gives an ample factor of safety against any possible emergency. From an aesthetic standpoint reinforced concrete bridges possess all the advantages of masonry bridges. In many cases more slender sections are employed, giving more graceful and pleasing lines, without any loss of strength. The cost of the reinforced concrete bridge in almost all cases will be much less than that of a masonry structure, and in many cases will not greatly, if at all, exceed that of a steel structure. They are free from the excessive vibrations often experienced in metal bridges, and if the foundations are protected against scour will withstand almost any flood and are proof against destruction by fire or tornadoes. The cost of maintenance is practically nothing, in the case of highway bridges being confined to keeping the pavements in repair.

The materials and labor used in the construction of this type of bridge are usually obtained in the locality of the bridge site and a large part of the money expended in the construction of the bridge is disbursed among the home people who pay for and use the bridge.

The girder bridge, on account of its freedom from corrosion, light weight and low cost, is particularly well adapted to the construction of foot bridges, and light highway bridges spanning railroads, canals and small streams. For short spans the flat slab answers all purposes, but for spans from 20 to 60 ft. and even up to 80 or 100 ft. the ribbed slab gives a satisfactory form of construction.

As a rule for spans greater than 50 feet the arch will prove the more economic form of construction.

The longest span reinforced concrete arch thus far built is that of the Gruenwald Bridge, at Munich, Germany. This bridge has two arched spans of 230 ft. This bridge is described on page 789. Mr. Edwin Thacher states that he can see no good reason why reinforced concrete bridges with spans of 500 ft. or more cannot be built with perfect safety, and often with economy; that he has designed and submitted bids on spans as great as 300 ft., and although the plans and prices were satisfactory, other and weightier considerations from the point of view taken by the officials induced them to prefer steel structures.

In this country fixed or monolithic arched bridges have been used almost exclusively, but in Europe both one and three hinged arches of reinforced concrete have been used for wide span bridges. In arched bridges the use of reinforced concrete is not confined to reinforcing the arch ring, but when solid spandrels are not employed it is used for the spandrel arches or posts and girders supporting a reinforced slab carrying the roadway. Again reinforced concrete may be used with economy in the construction of piers, abutments and abutment wing walls. It is also used for railroad trestles, the bents and girders being entirely of this material.

Cost.—In the construction of highway bridges light metal bridges with wooden floors will usually be found to be cheaper than reinforced concrete, but if heavy steel construction with trough floors be used the reinforced concrete bridge will prove to be the cheaper when no special difficulty is experienced in securing good foundations. When the questions of maintenance and permanency are considered the reinforced concrete bridge, it is believed, will prove the cheaper. A reinforced concrete viaduct at Rotterdam was constructed with a saving of 30 per cent. of the cost of a steel viaduct. In addition to this an income is obtained from the rental of shops built in the space under the viaduct. This space could not have been utilized in this way had a steel viaduct been constructed on account of the great noise due to the traffic on a metal structure. The contract for the Richmond and Chesapeake viaduct, Richmond, Va., described on another page in this chapter, was secured in competition with a steel structure, the question of cost alone determining the award of the contract. For railroad bridges of moderate span reinforced concrete supplies a material of construction which will give a permanent way and practically eliminate from the fixed maintenance charges the expense of a corps of bridge carpenters and inspectors. Again the use of trough floors of reinforced concrete filled with ordinary ballast permits the use of ordinary cross-ties on the bridges, thereby reducing maintenance charges, as bridge-ties are much more expensive than ordinary cross-ties. The use of ballast over bridges eliminates the train shocks so unpleasantly experienced when a train passes upon and leaves a metal bridge with framed bridge ties.

Of course local conditions will determine whether a steel or reinforced concrete structure should be used. It is probable that for spans of from 25 to 100 ft. the reinforced concrete arch will prove an economic structure. Many of the reinforced concrete arch bridges used on American railroads have been so designed that the concrete alone has sufficient strength to carry all loads, the steel being put in for additional strength because of ignorance of arch design. These structures cannot be said to be designed as reinforced concrete bridges, and have very high factors of safety. While not advocating the extremely light sections used in some reinforced concrete bridges of European design, the author believes that with careful designing lighter structures of ample strength may be secured and dollars saved for the railroad companies.

GIRDER BRIDGES.

For short span girder bridges where a reinforced slab without strengthening ribs is used the slab reinforcement may consist of any one of the systems of reinforcement already described for floor slabs. When heavy loads are to be carried care should be taken to see that there is sufficient concrete and metal provided to care for shearing stresses, as with short spans and heavy loads this will be found in many cases to be the determining factor in the design. For ribbed girder bridges any one of the systems of girder reinforcements already described for girder construction may be used. The methods to be used in the computations are the same as those employed for floor and beam design. In many cases, however, it would seem advisable in the design to treat the beams or ribs as if they acted independently of the slab, so proportioning them that they will have sufficient strength to carry all loads without assistance from the slab concrete, which in floor design is considered as acting with the rib to form a T-shaped beam. This seems desirable on account of the uncertain character of the load to be brought upon the bridge, the rough usage which it is at times subjected to, as well as being in many cases subjected to the action of ice and drift during time of flood.

For culvert and railroad bridges it is often impossible to design the bridge according to hard and fast rules, as the effect of impact from heavy trains is practically an unknown quantity. Under such conditions good judgment serves to fix the sections which should be used. For such bridges the cost of the extra materials necessary to give a well proportioned structure is of small moment, especially when it is remembered that the structure will form a permanent one with little or no expense necessary for repairs.

For highway and foot bridges conditions are different and theoretical designs may be more closely followed.

Beam Bridges.—Many short bridges of concrete and steel have been constructed for both highway and railroad spans. These

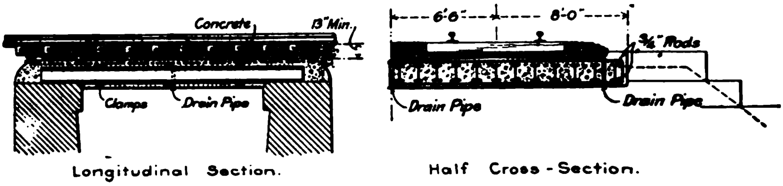


Fig. 568.—Steel Beam Bridge, B. & O. R. R.

consist either of longitudinal I-beams embedded in a concrete slab or longitudinal beams having jack arches of concrete sprung between them. The bridges shown in Figs. 568 and 569 are of this type and are standard bridges used on the Baltimore and Ohio Railroad. Simple slabs of concrete 12 in. thick reinforced with rails, as shown in Fig. 569, are used for spans from 5 to 12 ft. The concrete is made 1 : 3 : 5 with Portland cement and 1½ in. broken stone well rammed. Under each rail is embedded

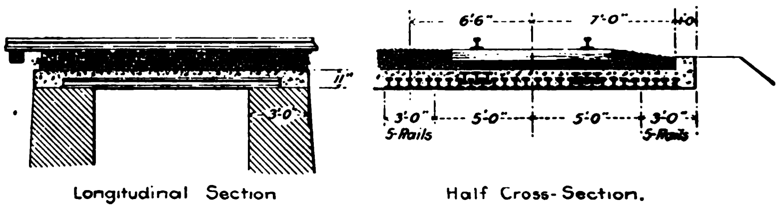


Fig. 569.—Steel Rail Bridge, B. & O. R. R.

a double line of rails set close together. The top line of rails have their heads turned down and placed between the webs of the lower line and the spaces between the rails filled with a 1:3 cement mortar. The rails are given a bearing of 18 ins. on the tops of the side walls.

For spans varying from 12 to 26 ft. steel beams are used in place of rails, as shown in Fig. 568. These beams have a bearing of 2 ft. on the masonry and are embedded in a mass of concrete, extending 2 ins. below and from 4 to 5 ins. above the top flanges. In this case a 1:2:4 concrete is used with 1-in. broken stone. The bottom sides and upper surfaces are finished with ½-in. of 1:3 mortar carefully rammed in with the concrete.

Figure 570 shows cross section of a beam bridge consisting of two 18-in. and three 20-in. I-beams. having concrete arches reinforced with expanded metal sprung between their bottom flanges. This bridge has a span of 28-ft. and was designed to carry a 12-ton roller.

Hennebique Bridges.—In Europe the Hennebique construction has perhaps been most extensively used for girder bridges This construction usually consists of a flat deck slab supported by longitudinal ribs or girders usually spaced from 3½ to 7 ft on centers, the whole being built as a monolith The slab is reinforced in the usual manner with straight and bent rods placed

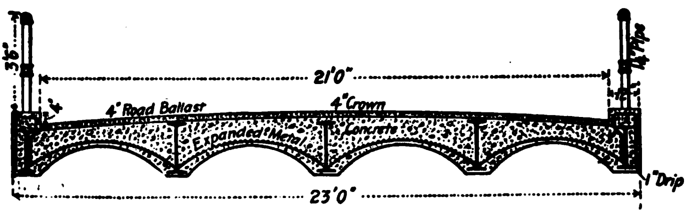


Fig 570.—Concrete and Steel Beam Highway Bridge.

transversely to the direction of the girder and has the usual Hennebique stirrups. The girders are reinforced in the usual manner with straight and bent rods and stirrups. The floor slabs are at times cantilevered from the outside girders to form a sidewalk, the reinforcing metal being properly placed to care for tensile stresses. The Sutton Drain and Milan bridges, described further on, are examples of Hennebique construction.

Moller Bridges.—In Germany the Möller ribbed slab construction is extensively used for girder bridges. The construction is similar in all respects to that used for floor slabs, the ribs being thicker at the center of the span, giving the fish-belly type. The reinforcement is usually a flat bar, having short angles riveted to it at intervals to anchor it in the concrete. (See Fig. 79, page 226). The reinforcement of the deck slab usually consists of small I-beams placed transversely to the ribs or girders. Figure 571 shows the principal features of the Möller construction.

Sutton Drain Bridge, Hull, Eng.—This bridge is of Hennebique construction and is the first reinforced concrete highway bridge constructed in England. The bridge is on a slight skew and has a square span of 40 ft. and a width of 60 ft between parapets, the roadway being 40 ft. in width and the two sidewalks each 10 ft. This bridge was designed to carry at the same time four wagons, each carrying 25 tons on two axles 8 ft. apart. The reinforced floor slab is carried by eight longitudinal beams 16 ins. wide and 2 ft. 7 ins. deep below the bottom of the slab. Three

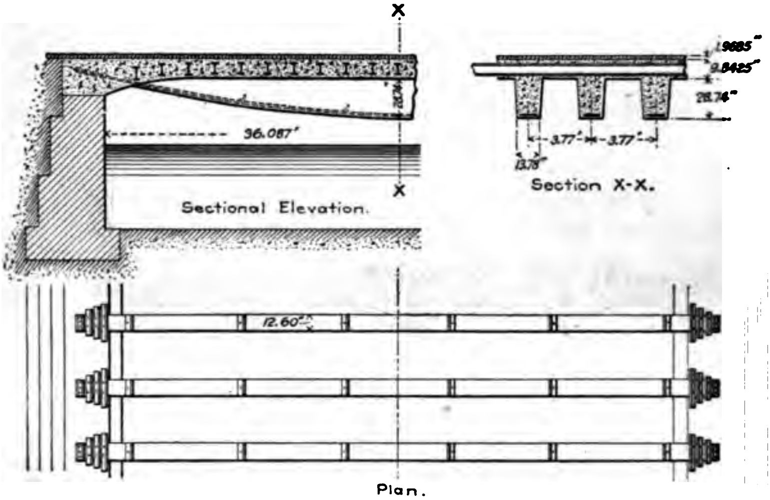


Fig. 571.—Moller Girder Bridge.

cross-beams, 8 ins. wide and 10 ins. deep, below the floor slab, span between the main beams under the roadway Three cross-beams, 8 ins. wide and 6 ins. deep connect the bottoms of the main beams under the sidewalk slab. These beams are intended to carry water and gas pipes across the bridge. Details of construction are shown in Fig. 572. The main beams are reinforced with four straight and four bent rods, 1¾ ins. diameter near the bottom and with two sets of four straight rods 1¼ ins. diameter near the top of the beam.

Skew Bridge at Milan.—The Milan bridge has an 83 ft. 8-in. center span and two 34 ft. 9-in. side spans measured on the skew, the skew angle being 65°. The piers are 6 ft. 7 ins. thick and are also of reinforced concrete. The clear width of bridge between parapets is 23 ft., including two sidewalks, 3 ft. 3 ins. wide Two main girders, which also act as parapets, are 2 ft. thick and 6 ft. 7 in. deep and extend 3 ft. 3 ins. above the foot paths. Reinforcement is placed in both the tension and the compression

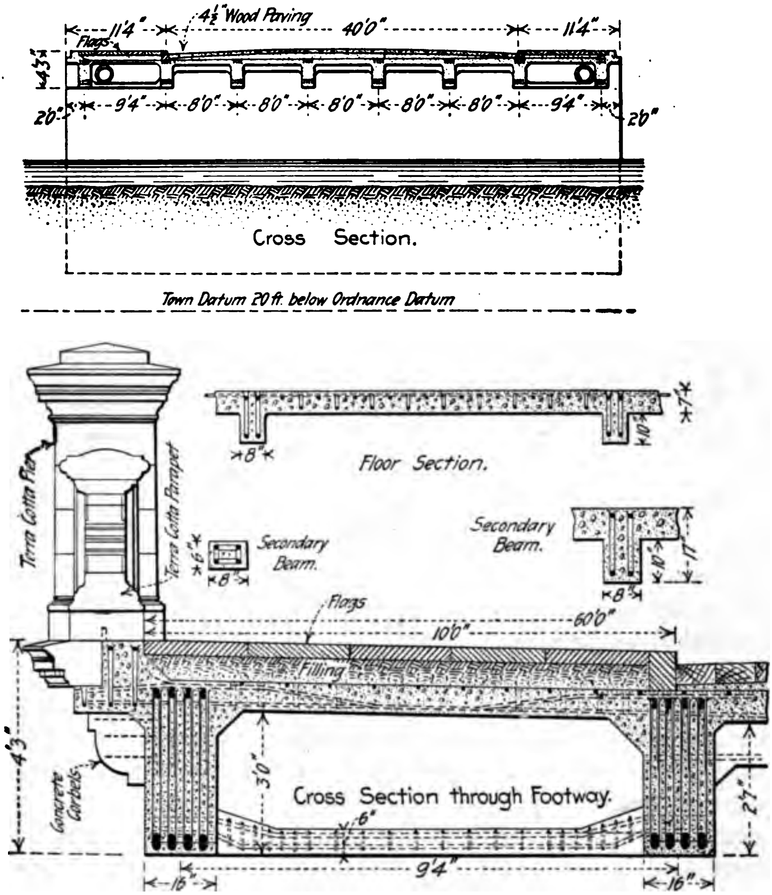


Fig. 572.—Hennebique Girder Bridge, Sutton Drain, England.

flanges of the beams. The compression reinforcement consists of 22 rods, approximately 1⅞ in. in diameter, while the tension reinforcement consists of 12 rods approximately 2⅝ ins. in diameter. A series of small rods is placed transversely between the longitudinal rods to tie them together. Cross-beams, approximately 10 ins. wide and 16 ins. deep below the deck slab, spaced 6 ft. centers, span between the main girders and support the deck slab, which is 5½ ins. thick. These beams are reinforced with three ⅝-in. rods at the top and six 1⅜-in. rods at the bottom. The deck slab is reinforced with both longitudinal and transverse rods. A partial elevation and cross sections showing details of construction are given in Fig. 573.

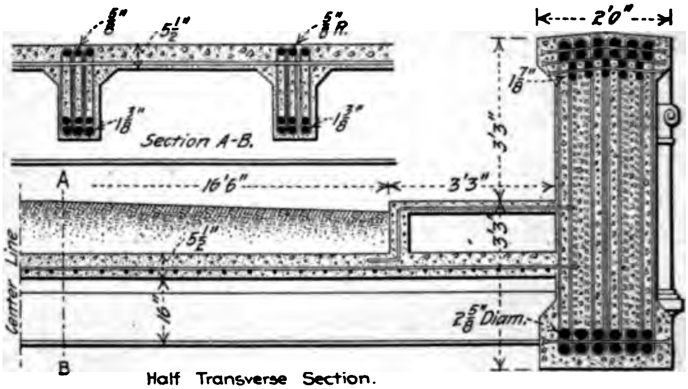


Fig. 573.—Details of Girder Bridge at Milan.

Albany, Ind., Bridge.—In one or more systems of ribbed slab construction an attempt has been made to secure a beam or rib of uniform strength by varying the depth of the beam—i. e., where the beam is uniformly loaded the cross section will increase from the abutment to the center directly as the bending moment of the beam increases

The Möller construction described on page 757 shows one method of applying this principle. This construction has been used both for floor and for girder bridge construction. The Albany, Ind., bridge is another example. This bridge was designed to secure a maximum economy of materials in order to meet steel bridge competition and also to secure a girder construction of uniform strength. The span is 40 ft. and the width of roadway 14 ft. The construction consists of an arched slab 8 ins. thick at the crown and slightly thicker at the ends and having a total rise of only 28 ins. The slab is reinforced with five longitudinal ribs, straight on the bottom and 28 ins. deep at midspan. No dependence is placed on the arched slab for carrying either dead or live loads, it being considered as acting only as a ribbed slab. The slab is reinforced with ⅝-in. diameter plain steel rods placed near its lower surface transversely to the axis of the roadway and spaced about 3 ft. centers. The ribs or girders are each reinforced with plain steel rods, having a section of 8 sq. ins. On account of the beam having a section to secure uniform strength the rods are equally stressed throughout the whole length of the span. This necessitates a positive anchorage at the ends having sufficient strength to develop the strength of the reinforcement. If the latter is bent sharply at right angles the concrete will crush under the rods; but by bending the rods with a sufficiently large radius this is avoided, and by gradually reducing the radius of curvature a sufficient length of rod will be embedded in the concrete to develop their strength by means of the adhesion and friction on the rod. The method of bending the rods to secure a sufficient anchorage is shown in

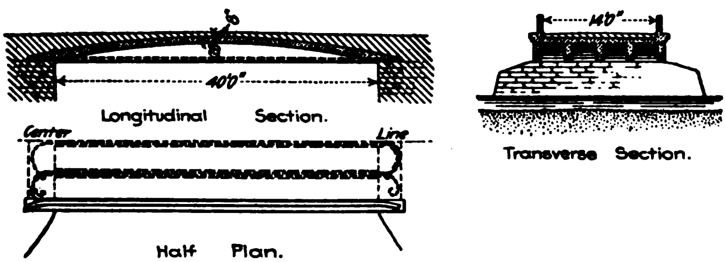


Fig. 574.—Girder Bridge at Albany, Ind.

Fig. 574, which is a half-plan longitudinal and transverse section. This bridge was designed to carry a 20-ton concentrated load, together with 200 lbs. per sq. ft. uniform live load, besides the dead weight of bridge and the filling upon it.

Elmwood Bridge, Memphis, Tenn.—The 100-ft. reinforced concrete girder bridge recently constructed at Memphis, Tenn., is the longest span highway girder bridge thus far constructed. The necessity of providing a clearance of at least 19 ft. over the six railroad tracks spanned by this bridge and the strong objections raised to a graded approach which would have been required in the cemetery ground if an arch span with the required clearance had been used, made the adoption of a practically flat span imperative. The bridge as built has a rise of 4 ft., but was not designed to act as an arch. Two longitudinal girders are used to carry the reinforced concrete slab, which is suspended from them and forms the roadway and sidewalk. The bridge has a total width of 31 ft., there being a 16-ft. roadway, two 4-ft. sidewalks and the two 3-ft. 6-in girders between the roadway and sidewalks. The girders have a total height of 6 ft. 6 ins., including a 6-in. coping. Massive concrete abutments are provided at both ends and the girders are designed to act as a fixed or restrained beam. Under these conditions the portions of the girders near the abutments act as cantilever beams, while the middle portion acts as a simple beam. The reinforcement of the cantilever portion consists of 30 bars, 1½-in. diameter, placed in four horizontal rows in the upper or tension portion of the beam. These bars are 40 ft. long, extend 4 ft. beyond the inflection points or ends of cantilever sections of the girder and extend 15 ft. back into the abutment. The middle portion of the girder, which acts as a simple beam, is reinforced near the bottom with 24 bars 1½ in. diameter placed in three rows: This reinforcement is 66 ft. long and extends 4 ft. into the cantilever section at each end, i. e., the two systems of rods lap 4 ft. at each end of the girder. At these points 30 inclined shearing rods, 1½-in. diameter, transmit the shear from the lower to the upper rods. Each cantilever is anchored to the abutment by a series of 1½-in. diameter rods running from a central point at the top back and downward into the abutment. Several light street railway rails were placed in a vertical position near the end of each girder to bond the girders and abutment together.

The abutments were designed to act as anchorages for the cantilever girders, any possible thrust due to arch action being neglected. Between the girders the abutments are hollow, having a retaining wall at the front and a wide flat floor cantilevered back from the rear face of the wall. This floor is reinforced near its lower surface with light rails. The front wall retains the earth, which resting upon the floor provides additional anchorage. The bridge floor is 13 ins. thick and is reinforced with 10-in. tranverse I-beams. Each beam is attached to the girders on their center lines by two 1-in. tie rods extending up into the girder and anchored just below the coping on the latter by means of a ½ × 6-in. steel bar embedded in the concrete. Details of construction are shown in Fig. 575.

This bridge is designed to sustain a uniform live load of 200 lbs. per square foot. The dead load is approximately 500 tons.

The false work used in the erection of this bridge consisted of bents built of 12 × 12-in. timbers heavily cross-braced and supporting 5 × 16-in. stringers, which carried the lagging.

The concrete was a 1 : 2½ : 5 Portland cement broken stone concrete mixed fairly wet. Corrugated bars were used for the reinforcement. Each girder was built in a single day. The forms were not removed until three months after the concrete had set.

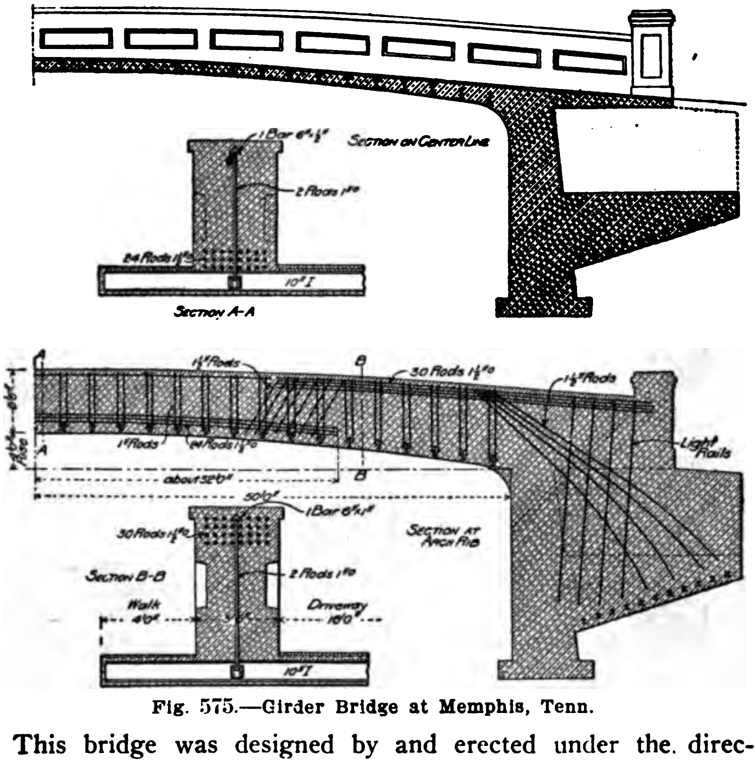


Fig. 575.—Girder Bridge at Memphis, Tenn.

This bridge was designed by and erected under the direction of Mr. J. A. Omberg, City Engineer of Memphis, who states that the total cost of the bridge was $17,500, including the asphalt pavement, iron hand railing and cutstone veneering on posts at the end of the girders. The forms cost $4,000. This large cost was due to difficulties in erection, which was done so as not to interfere with traffic.

Open Web Girder Bridge.—The bridge at Purfleet, England, **differs from** the usual type of girder bridge in that it is constructed with open webs. On account of limited head room it was necessary to use a through bridge. The bridge rests upon circular piers and forms a part of an approach to an island pier.

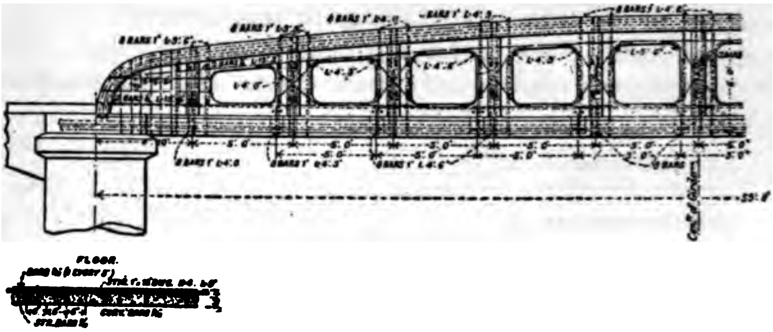


Fig. 576.—Half Elevation, Open Web Girder Bridge, Purfleet, England.

It is used to carry a single line of standard gauge railway. The main portion of the pier also consisted of reinforced concrete piles and girders. On account of this bridge being built on a

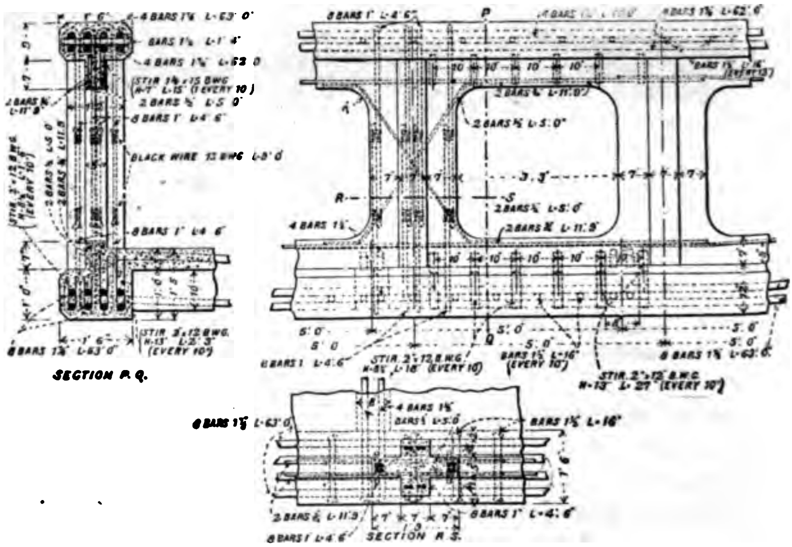


Fig. 577.—Details of One Panel, Purfleet Bridge.

sharp skew, it has an out to out span of 59 ft. 8 ins. and a square span of 35 ft. Figure 576 shows a half elevation of one truss, while Fig. 577 shows sections and elevation of one panel of the truss. The main portion of the top chord is 9x18 ins. in section, and is reinforced with eight 1⅞-in. round bars; cross bars 1½ in. in diameter, 1 ft. 4 ins. long, are inserted transversely at intervals to prevent the concrete from bulging laterally when under high stress. The reinforcement of the bottom or tension chord also consists of eight round rods 1⅞ ins. diameter, bound together by stirrups of No. 12 S.W.G. iron 2 ins. wide, spaced about 10 ins. centers. The vertical members are cruciform in cross-section, each arm of the cross being 7 x 5 ins. in section. They are reinforced with vertical bars hooked over the bottom and top chord bars, and with diagonal bars bent

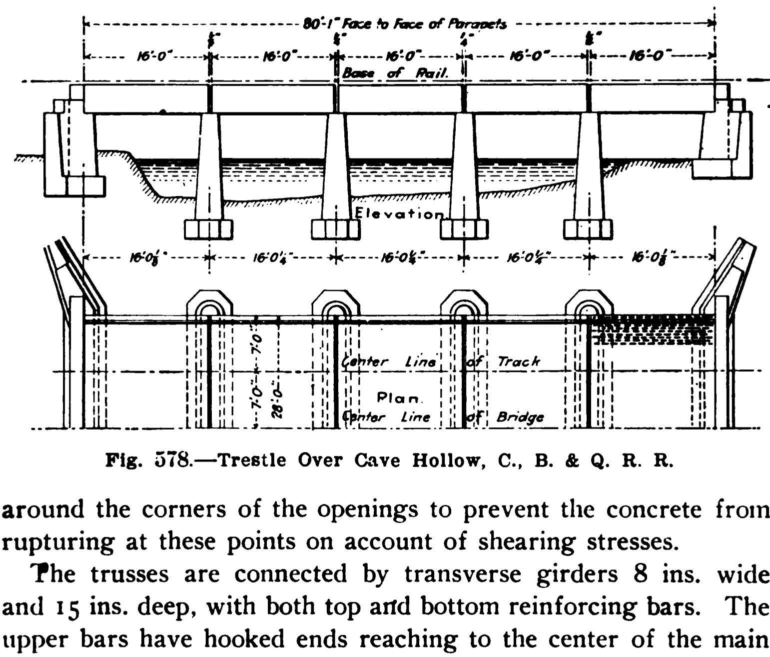


Fig. 578.—Trestle Over Cave Hollow, C., B. & Q. R. R.

around the corners of the openings to prevent the concrete from rupturing at these points on account of shearing stresses.

The trusses are connected by transverse girders 8 ins. wide and 15 ins. deep, with both top and bottom reinforcing bars. The upper bars have hooked ends reaching to the center of the main truss section, while the lower bars are carried through the chord between the two rows of 1⅞-in. bars forming the reinforcement of the main trusses. The floor of the bridge consists of a reinforced slab 5 ins. thick, stiffened with longitudinal ribs placed directly under the track rails. This bridge is of Hennebique construction, and all reinforcing members are anchored in the concrete, and the whole firmly bound together by numerous sheet iron stirrups. It is stated that the structure weighed 90 tons, of which 15 tons is the weight of the reinforcement. The concrete was composed of 1 part Portland cement and 4 parts crushed stone, aggregate all passing a ¾-in. screen. This bridge when tested by two cars with bogie trucks and 24 ft. wheel base, loaded to 30 tons each, showed a maximum deflection of 0.197 ins.

The form given to the main trusses would seem to make a determination of the stresses therein impossible, a state of affairs

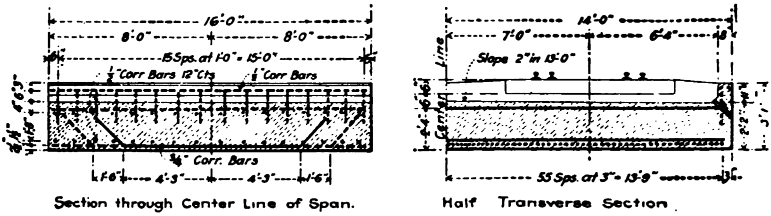


Fig. 579.—Details of Floor, Cave Hollow Bridge.

which, as far as possible, should be avoided in bridge or structural design.

Viaducts.—Viaducts of reinforced concrete have been used to a limited extent both in this country and in Europe. Fig. 578 shows a half plan and elevation of a reinforced concrete trestle of heavy construction over Cave Hollow, on the Chicago, Burlington & Quincy Railway. Fig. 579 shows longitudinal and trans-

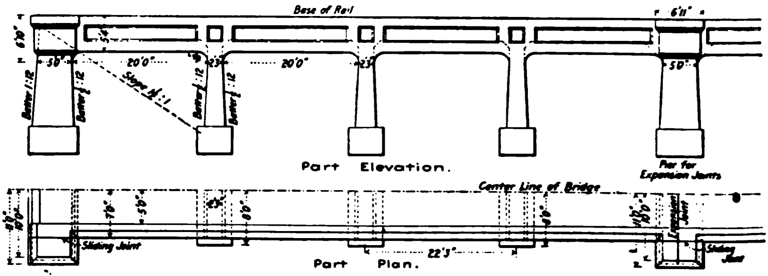


Fig. 580.—Viaduct for C., C., C. & St. L. Ry.

verse sections, with details of construction. The longitudinal reinforcement consists of ¾-in. corrugated bars spaced 3 ins. centers, while the transverse bars are ½ in. spaced 12 ins. centers.

Cleveland, Cincinnati, Chicago & St. Louis R. R. Viaduct.—Several viaducts of reinforced concrete have been recently constructed by the C., C., C. & St. L. R. R. Fig. 580 shows an elevation and a partial plan of a portion of one of these viaducts

Fig. 581 shows a longitudinal and a transverse section of the piers, floors and girders. These structures consist of a series of piers of reinforced concrete supporting girders of 20 ft. clear span. The longest viaduct constructed has a total length of 1,217 ft. The girders are 5 ft. 4 ins. deep and 2 ft. wide, and are connected at mid-depth by a flat floor slab 1 ft. 9 ins. thick. The arrangement of the reinforcing bars is clearly shown. For the sake of appearance the faces of the girders and piers are relieved with paneling, as shown on the elevation. A horizontal expansion joint is provided on each abutment and on every fourth pier. Move-

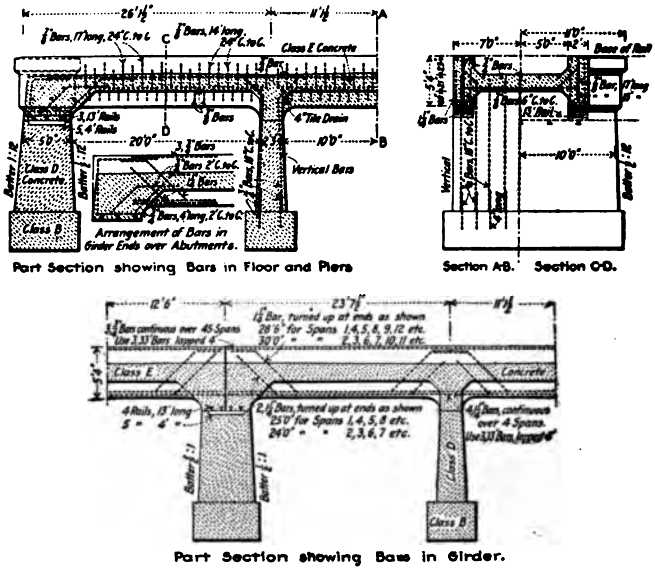
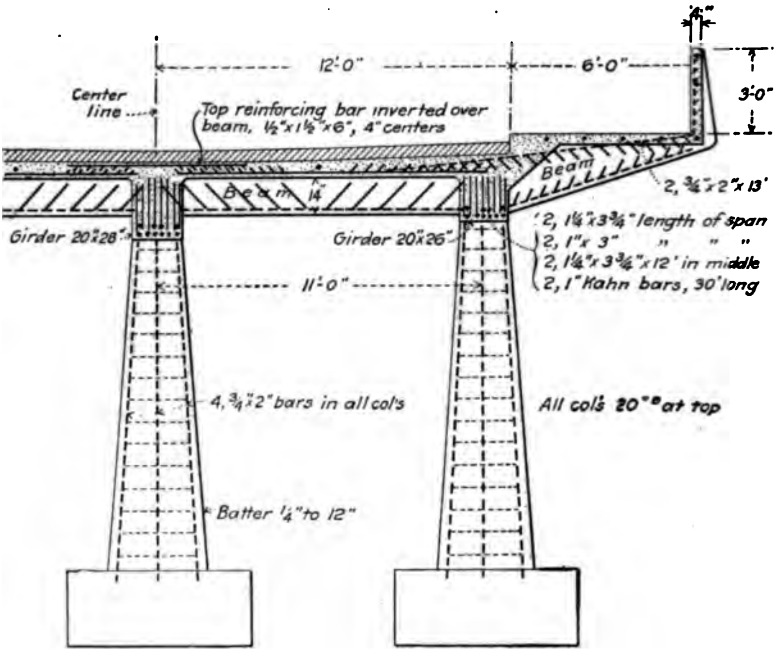


Fig. 581.—Details of Viaduct Shown in Fig. 580.

ment is provided for by transverse inverted rails embedded in the upper portion, resting upon short lengths of rails embedded in the concrete of the pier itself. To insure clean contact between rails those of each course project ⅛ in. from the face of the concrete, forming a ¼-in. open joint between the girder and the pier, which is filled with layers of felt. At each fourth pier also there is a vertical expansion joint in the floor and girder. This is filled with two layers of felt reaching to within 2 ins. of the top, the upper portion being filled with asphalt.

Fig. 582 shows a partial cross-section of a viaduct approach recently constructed at Oklahoma City, Okla. T. The piers forming the bents were spaced 26 ft. centers. Three longitudinal girders, one 20 × 28 ins. and two 20 × 26 ins. in section, span between bents and carry the roadway slab. The columns are 20 × 20 ins. at the top and increase in size with a batter of ¼ in. to the foot downward. The general details, size and amount of reinforcement are shown on the drawing.

Guadalquiver River Viaduct.—Two viaducts between 30 and 40 ft. in height were recently constructed for a mineral railway



Half Section with Walk.

Fig. 582.—Viaduct Approach to Bridge at Oklahoma City, Oklahoma.

near Seville, Spain. One viaduct has a length of 371 ft. and the other of 284 ft. The scheme for handling cars to and from a loading pier at the Guadalquiver River, where the ore is loaded in barges by tipping the cars, involves two tracks, the cars being pushed to the loading pier by a locomotive and run back by gravity. The two tracks are carried on separate lines of girders supported by double bents braced together transversely. Figure 583 shows an elevation of the shorter viaduct and loading pier; Fig. 584 shows a transverse section through the

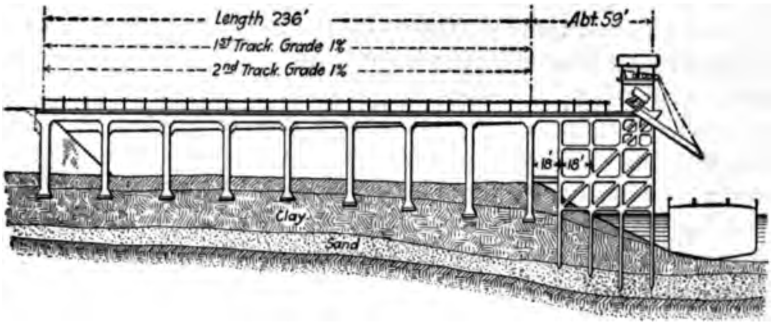


Fig. 583.—Elevation of Viaduct and Loading Pier, Guadalquiver River, Spain.

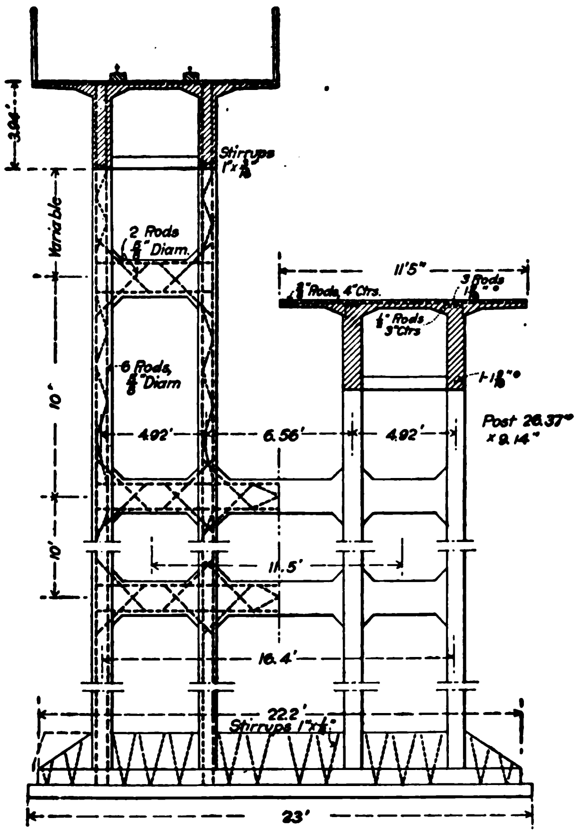


Fig. 584.—Transverse Section, Guadalquiver River Viaduct.

bents and Fig. 585 the detail of the girder reinforcement. The bents are spaced approximately 29½ ft. centers, and are composed of four posts approximately 26 x 9 ins. The girders spanning between the bents are about 48 x 9 ins. in section and carry a slab which supports the track and is cantilevered out to carry the sidewalks. Two or more struts brace the posts forming the bents in a transverse direction. Fig. 584 also shows the footing for the bent posts; this consists of a slab 5 ins. thick, 5 ft. 9 ins. wide and 23 ft. long, stiffened by a rib between the posts. This reinforced footing rests upon a bed of concrete about 8 ins. thick. The maximum pressure brought upon the soil is about 1.2 tons. The train loads for which this viaduct was designed consists of a 36-ton locomotive on coupled axles about 9 ft.

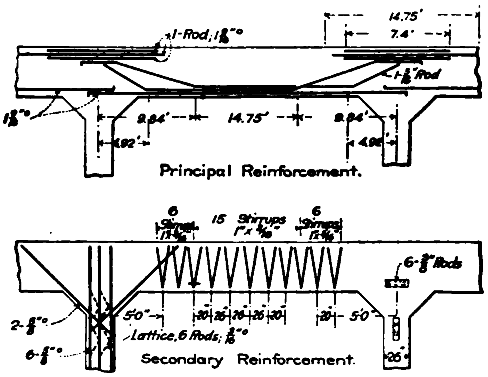


Fig. 585.—Girder Reinforcement, Guadalquiver River Viaduct.

centers and 12 tons on a bogie truck with axles 5 ft. 3 ins. centers and 20 cars carrying 20 tons each on axles 5 ft. 3 ins. apart.

It will be noted that no longitudinal bracing is used between the bents, the only stiffness in this direction being that of a column 26 ins. wide, which for a height of 30 or 40 feet would appear to be beyond what is usually considered a safe limit. For the short trestle, which is only 284 ft. long, the braced loading pier at one end and the bank at the other will prevent longitudinal movement. For the longer trestle, which is on a curve, it would appear that some form of longitudinal bracing should have been used.

Richmond & Chesapeake Bay Ry. Co.'s Viaduct.—A reinforced concrete viaduct having a total length of about 3,000 ft. and a maximum height of 65 ft. has been recently constructed at

Richmond, Va. This structure is of reinforced concrete throughout, and was designed and the construction supervised by the Trussed Concrete Steel Co., of Detroit, Mich. The Kahn bar was used for reinforcement throughout. The construction consists of reinforced concrete girders from 23 to 49 ft. long, supported by bents and towers of reinforced concrete.

The 23-ft. girders are 12 × 30 ins. in section, and are connected and braced by transverse struts 3½ × 18 ins. in section. These struts are flared at the ends and divide the floor into three parts, having irregular octagonal openings 5 ft. 9 ins. in width. Details of the standard 23-ft. girder spanning between the single post bents used for heights up to about 21 ft. are shown both in plan and elevation in Fig. 586. The sizes and location of reinforcement for girders, struts and posts are clearly shown in this figure.

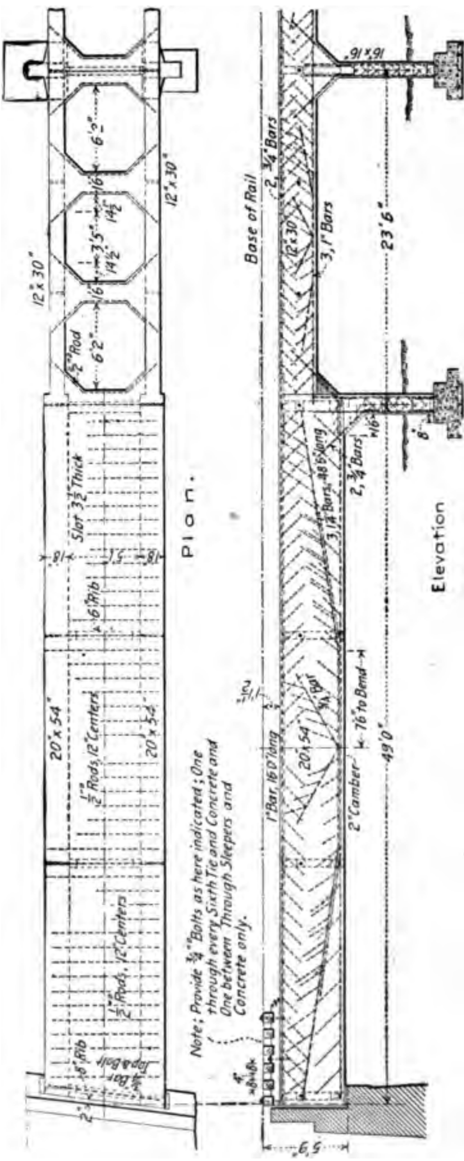


Fig. 586.—Plan and Elevation of 23-Ft. and 49-Ft. Spans, Richmond Viaduct.

A transverse section through the girder and bent is shown in Fig. 587, together with general features of construction. The cross-section of the standard post used for this bent is shown in Fig. 588. As will be seen, the post is 16 ins. square and is reinforced with four ¾-in. Kahn bars tied together every 14 ins. with ⅜-in.-diameter rods. The detail of the column bases, as should be noted, are also of reinforced concrete.

The 12 × 30 in. girders are reinforced with three 1-in. bars 23 ft. long and two ¾-in. bars 16 ft. long at the middle. Two ¾-in. bars are also placed near the top of the beam. The cross struts are reinforced with ½-in. round rods.

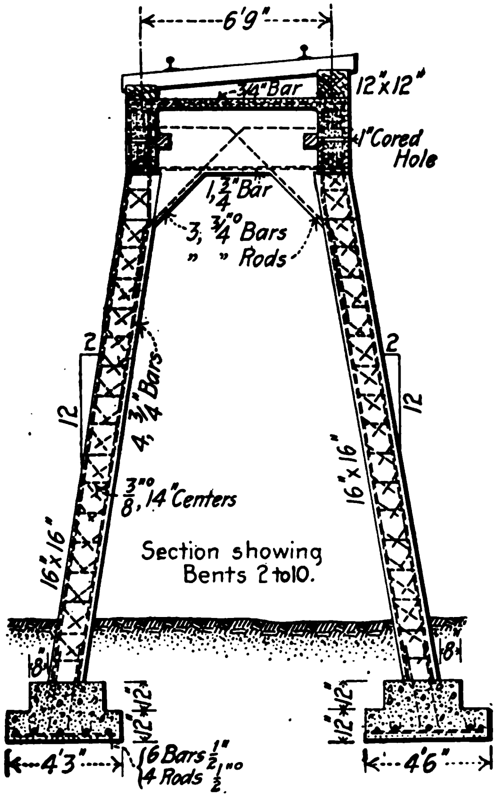


Fig. 587.—Details of Bents Nos. 2 to 10.

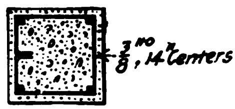


Fig. 588.—Section Through Typical Column.

As shown in Figs. 586 and 589, occasional spans have the girders connected with a full top slab. This gives additional stiffness, and when the girders cross a street, as in girders between bents 44-45, and bents 48-49, Fig. 589, protects the street below.

Details of a 49 ft. girder span are also shown in Fig. 586. These girders are 20 × 54-in. in section and are connected by

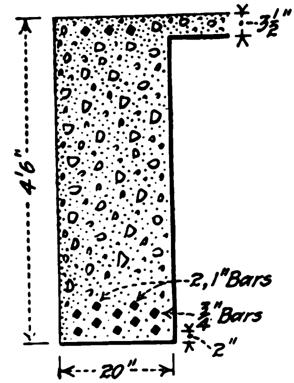


Fig. 590.—Section of 20 x 54-In. Girder.

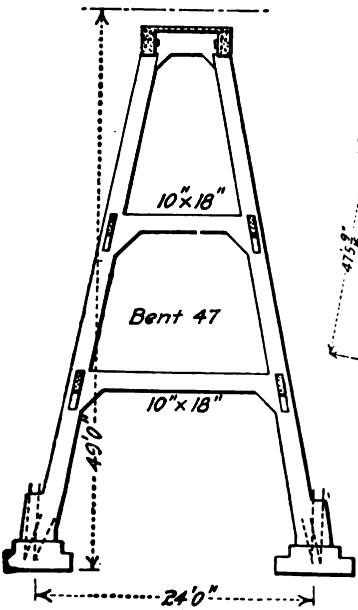


Fig. 591.—Typical Trestle Bent.

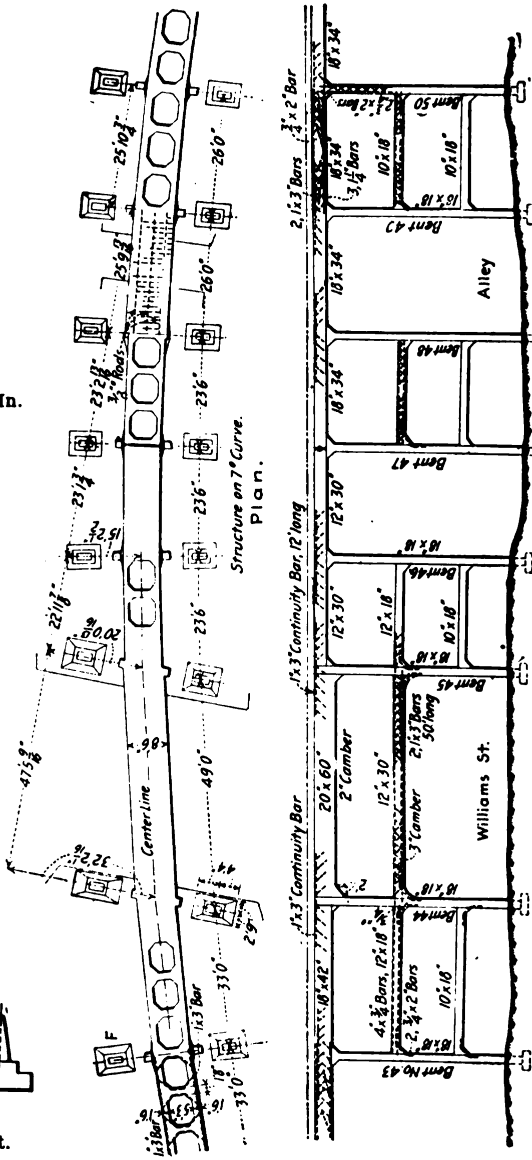


Fig. 589.—Plan and Elevation, Showing Construction of Richmond Viaduct.

a continuous 3½-in. deck slab. The cross-section of this girder is shown in Fig. 590. Two cross ribs, as shown in Fig. 586, brace the girders transversely. The 3½-in. deck slab is reinforced by one ⅜-in. diameter rod running longitudinally along its center line and ½-in. diameter rods spaced 12-in. centers and running transversely.

The general arrangement of the towers will be understood

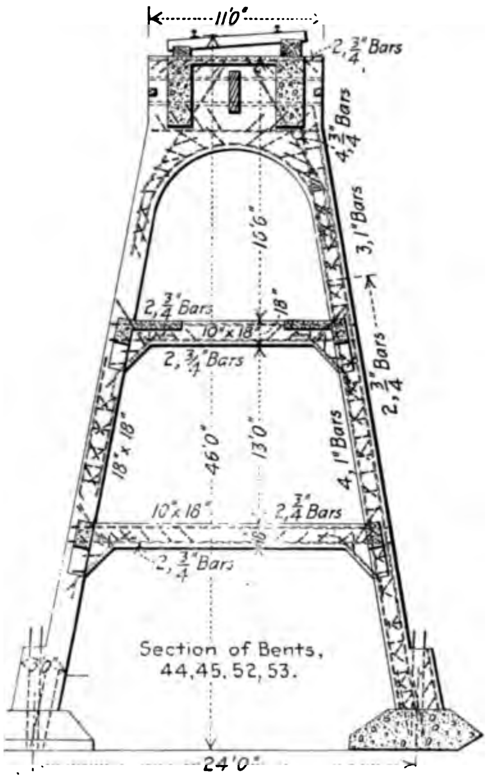


Fig 592.—Details of Tower Bent.

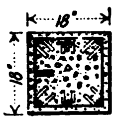
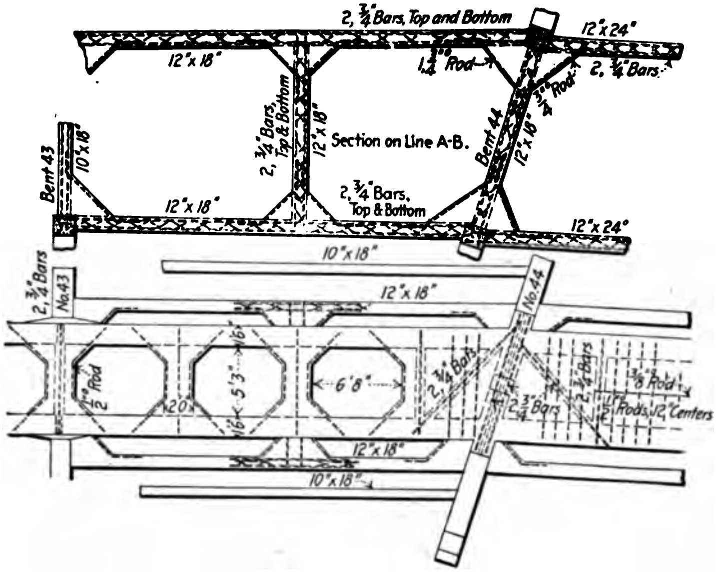


Fig 593.—Typical Column Section.

from Fig. 589. Two or more separate towers consisting of bents connected by longitudinal struts alternate with two towers connected by longer span girders as shown in the left-hand portion of the figure. Fig. 591 shows a typical bent, while Fig. 592 shows details of the tower bent used for the higher towers. For a height of 49 ft. 18 × 18-in. posts connected by two transverse and two longitudinal struts were used. The rein-



Figs. 594 and 595.—Plan and Horizontal Section on Line A-B, Tower No. 43-44.

forcement consists of six ¾-in. bars as shown in Fig. 593. A 20 × 20-in. post was used at the highest point of the trestle where the base of rail was 65 ft. above the surface of the ground. Fig. 594 shows section A-B through tower 43-44. The trans-

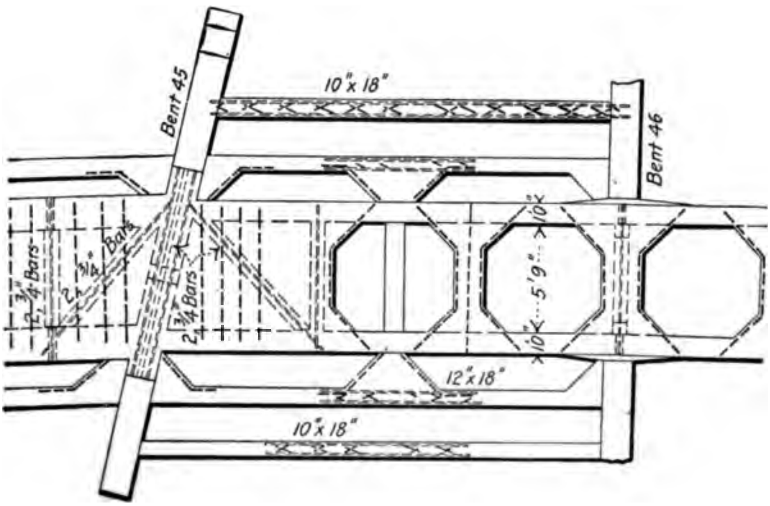


Fig. 596.—Plan of Tower No. 45-46.

verse and longitudinal bracing of this tower is shown in detail in this figure. Fig. 595 shows a plan of the same tower together with plan of girders and their bracing. The slender section of longitudinal strut between bents 43 and 44, viz.: 12 x 30-in. for a span of 49 ft., should be noted.

The plan of tower 45-46 is shown in Fig. 596. The method of stiffening this tower in the plane of the top of girders should be noted.

The details of reinforced concrete shoe marked "F" and used for outside post of bent 43 is shown in Fig. 597. The other

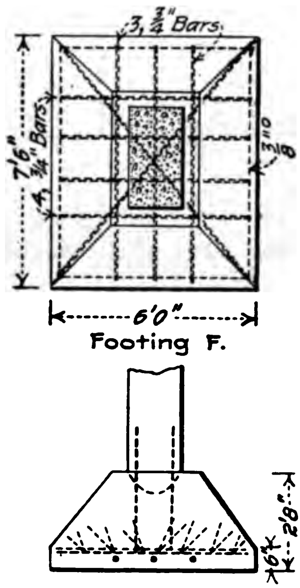


Fig. 597.—Typical Column Footing.

shoes were of similar construction, the amount and arrangement of reinforcement varying with the size of shoe needed.

Expansion joints were provided at suitable intervals. Fig. 598 shows plan of expansion joint at bent No. 5, and Fig. 599 shows clearly details of construction of expansion joint.

This structure was designed to carry a live load of 75 tons on two trucks 33 ft. apart, each truck consisting of two axles 7 ft. centers. It was assumed that the structure should carry its dead load, the full live load, and 50 per cent. of the live load for impact. Wind pressure was taken at 30 lbs. per square foot on surface of train and surface of structure. The longitudinal thrust due to braking of trains was taken as 20 per cent. of live load. The loading on towers due to centrifugal force of train on a 7 degree curve was taken as 14 per cent. of live load. As will be noted, the usual diagonal bracing used on all metal towers is replaced by transverse and longitudinal struts, the intention being to so design all joints and all members that they will possess the necessary rigidity to withstand all bending coming upon them. The success or failure of this viaduct will

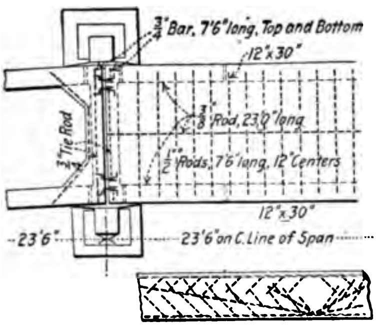


Fig. 598.—Plan Showing Expansion Joints.

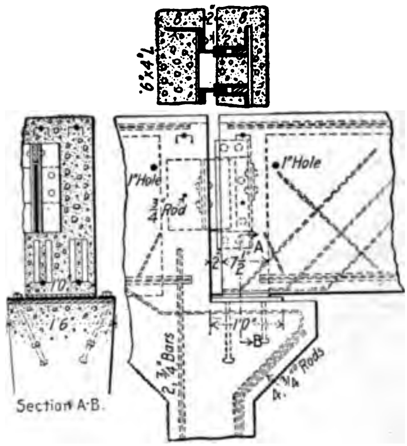


Fig. 599.—Details of Expansion Joints.

be watched with much interest, as it is the largest structure of this kind which has thus far been built of reinforced concrete.

ARCH BRIDGES.

Arch bridges of concrete are rapidly replacing masonry bridges and in many cases metal structures. Whether it is necessary or not to use steel reinforcement for the arch ring depends upon the conditions governing each bridge and the individual judgment of the designer. A number of railroad bridges have been built during the past few years in which the arch concrete was designed to carry all stresses without reinforcement and then metal added when the structure was built. While this practice certainly gives a safe bridge, it is a needless waste of materials and hardly conforms to the ethics of good engineering.

The principal methods of locating the metal reinforcement in a cross section of the arch formed by a plane cutting the arch ring at right angles to its axis have been shown in Figs. 145 to 154.

Arch Design.—The methods employed for the design of a stone masonry arch are usually employed in preparing the design for a plain concrete arch ring, and may also be employed in the design of reinforced concrete arches. The line of resistance of the arch is determined and located graphically on the profile of the assumed arch. If this line lies within the middle third of the arch ring, no provision for tension will be necessary, and if the assumed safe compressive stresses on the concrete are not exceeded the arch will be safe. It is customary to divide the arch ring by radial planes into sections of convenient length, such that the line of resistance will not depart far from the curve of the lineal arch, *i. e.*, the medial line of the assumed arch ring. When a reinforced arch is under consideration, if as is usually the case, the reinforcement is symmetrically located, the same method may be followed.

If the reinforcement is not symmetrical in the arch rib, considerable difficulty will be experienced in locating the neutral axis at any section. Again the position of the neutral axis undoubtedly shifts about under the action of different loads. The presence of steel in a rib will enable it to safely resist tension, but it is the practice of the best designers to so proportion the arch rib that it is seldom subjected to tension under dead and live loads. The maximum stress which can occur at any section of the arch is that due to the thrust of the arch acting along the line of resistance, the bending moment caused by such an arrangement of the loads as to produce the greatest possible stress upon the given plane and the bending moment due to temperature changes. It is usual not to exceed a unit pressure of about 500 lbs. per sq. in. on the concrete for dead and live loads. For temperature stresses, however, these compressive stresses are allowed to run up to from 650 to 800 lbs. per sq. in. These maximum stresses also include dead and live loads and wind stresses. When tension is allowed in the concrete, the unit tensile stress allowed varies from 50 to 75 lbs. per sq. in. When the bending moment at the given sections has been determined, the necessary concrete and steel sections may be computed.

For methods of determining the line of resistance, Baker's Masonry Construction or any well known work on graphic statics may be consulted. For determining the true line of resistance the external forces must be known or assumed, and the direction, point of application and amount of thrust at the crown shall be known. It is customary to assume, besides the usual dead loads, at least two live loadings: (1) that the arch carries the maximum live load over the entire span: (2) that the arch carries the maximum live load over one-half the span. The first of these loadings gives the maximum thrust and the second the maximum bending moment. Such sections should be chosen as to satisfy both conditions.

The elastic theory of the arch is to be preferred over the above method, as it gives a much more satisfactory analysis. A discussion of the elastic arch theory cannot be given in this place. Among the best treatises on this subject are, "Theory of Steel-Concrete Arches," by Prof. William Cain. "Symmetrical Masonry Arches," by Prof. Malverd A. Howe. "Trusses and Arches, Part III," by Prof. Charles E. Greene. The author prefers Prof. Greene's method as giving the most satisfactory solution.

Thacher's Formulas.—Having found the thrusts, bending moments and shears at chosen sections in the arch ring, the intensities of the stresses in the concrete and the metal and the necessary distribution of the material may be obtained by the use of Mr. Edwin Thacher's formulas. The author is indebted to Mr. Thacher for permission to publish these formulas. These formulas apply to any form or variety of reinforced concrete arch or beam within the limits of elasticity of concrete. The following nomenclature will be used:

E_c = modulus or coefficient of elasticity of concrete.
E_s = modulus or coefficient of elasticity of steel.
$\frac{E_s}{E_c} = e$.
A_c = area of section of concrete one inch wide, square inches.
A_s = area of section of steel in width b, square inches.
a = area of steel per inch width = $\frac{A_s}{b}$, square inches.
I_c = moment of inertia of concrete, A_c, about common neutral axis.
I_s = moment of inertia of steel, a, about common neutral axis.

f_c = intensity of stress in the concrete, lbs. per sq. in.
f_s = intensity of stress in the steel, lbs. per sq. in.
d = depth of concrete in inches.
d′ = depth of steel in inches.
u = distance from neutral axis of combination to outer fibre of concrete, in inches.
v = distance from neutral axis of combination to outer fibre of steel, in inches.
T = thrust on section of arch, one inch wide, in pounds.
M = bending moment on section one inch wide in foot pounds.
P = pressure on line of pressure on section one inch wide.
k = distance from neutral axis of combination to line of pressure in inches, taken normal to line of pressure.
c = distance from center of gravity of steel rib to bottom of concrete, in inches.
b = distance from center to center of steel ribs, measured in the direction of the width of the arch, in inches.

For sections in which the steel reinforcement is arranged symmetrically about the center of gravity of the concrete.

$$f_c = \frac{T}{A_c + ea} \pm \frac{6\,d\,M}{I_c + e\,I_s} \quad \ldots\ldots (1).$$

$$f_s = \frac{e\,T}{A_c + ea} \pm \frac{6\,e\,d'M}{I_c + e\,I_s} \quad \ldots\ldots (2).$$

For sections in which the steel reinforcement is not arranged symmetrically about the center of gravity of the concrete.

$$f_c = \frac{T}{A_c + ea} \pm \frac{P\,k\,u}{I_c + e\,I_s} \quad \ldots\ldots (3)$$

or,

$$f_c = \frac{T}{A_c + ea} \pm \frac{12\,u\,M}{I_c + e\,I_s} \quad \ldots\ldots (4).$$

$$f_s = \frac{e\,T}{A_c + ea} \pm \frac{e\,P\,k\,v}{I_c + e\,I_s} \quad \ldots\ldots (5).$$

or,

$$f_s = \frac{e\,T}{A_c + ea} \pm \frac{12\,e\,v\,M}{I_c + e\,I_s} \quad \ldots\ldots (6).$$

The distance of the neutral axis of the combination above the soffit of the arch or the bottom of the concrete will be

$$d - u = \frac{\frac{1}{2} A_c\,d + e\,a\,c}{A_c + e\,a} \quad \ldots\ldots (7).$$

It is Mr. Thacher's practice, and that of many other engineers, to require that sufficient steel shall be used to take the entire bending moment of the arch without aid from the concrete and not exceed the elastic limit of the steel. The steel can never take the entire bending moment unless the concrete fails. This cannot occur if the arch ring is so designed, as is the usual practice, that the line of pressure will fall within the middle third. To satisfy the condition that the steel within its elastic limit shall be capable of taking the entire bending moment, assuming the elastic limit of steel at 36,000 lbs. per sq. in., we have

$$I_s = \text{or} > \frac{M v}{3{,}000} \quad \ldots\ldots\ldots\ldots\ldots\ldots (8).$$

For symmetrical sections, *i. e.*, when the steel is symmetrically placed about the center of gravity of the concrete, we have

$$a = \text{or} > \frac{M}{1{,}500\ d'} \quad \ldots\ldots\ldots\ldots\ldots\ldots (9).$$

$$A_s = \text{or} > \frac{M b}{1{,}500\ d'} \quad \ldots\ldots\ldots\ldots\ldots\ldots (10).$$

At the crown also make

$$A_s = \text{or} > \frac{d b}{150} \quad \ldots\ldots\ldots\ldots\ldots\ldots (11).$$

Equation (11) is obtained as a result of Mr. Thacher's rule that the total cross-section of the steel at the crown shall not be less than 1-150 of the cross-section of the concrete at that point. Usually the area of steel falls within the limits 1-50 and 1-150 of the area of the concrete at the crown.

The value assumed for the value of $e = \frac{E_s}{E_c} = 20$ by many engineers enables the use of higher stresses in the steel than is possible when a lower value for e is used. The reason for using such a high value for e is that it is believed that the value of the modulus of elasticity E_c for concrete in large masses is lower than for concrete in small sections usually used for slabs and beams. Under the conditions obtaining in arches the modulus is taken at 1,500,000 lbs. per sq. in.

Austell Bridge.—The four-span reinforced concrete arch bridge of the Southern Ry. near Austell, Ga., is composed of four three center arch spans. The clear span is 70 ft. and the rise 20 ft. from spring line to intrados. The radii of the arch are 12 ft. 3 ins., 29 ft. and 56 ft., respectively. This bridge may be taken as an example of a heavy reinforced bridge of the Monier type, corrugated bars being used to form the reinforcement. The arches have a thickness of 3 ft. 4 ins. at the crown and have parapet walls 2 ft. 8 ins. high. Near the intrados the arch is reinforced longitudinally for 30 ft. with ¾-in. bars, 12 ins. on centers with a depth of cover of 4 ins., thence lapping 6 ft. and placed 5 ins. on centers; the reinforcement continues with 1¼-in. bars to the center of the arch, where the lap is 7 ft. Transverse bars of ½-in. section are placed 3 ft. centers. In the

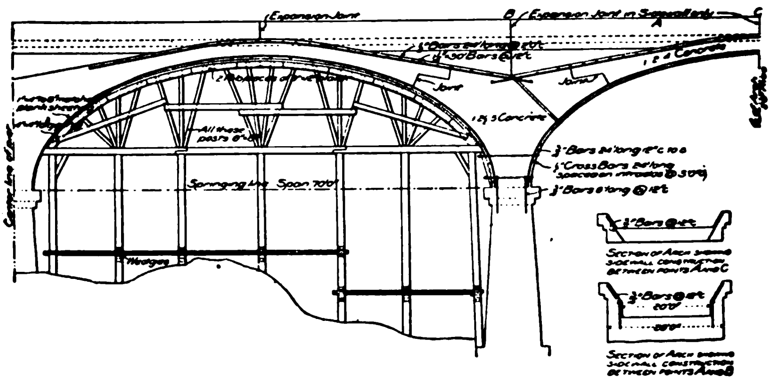


Fig. 600.—Southern Railway Bridge, Austell, Ga.

extrados the reinforcement consists of 1¼-in. bars throughout the entire length placed 12 ins. on centers. The lap of the extrados bars is 7 ft. The side walls are reinforced by ½ and ¾-in. bars bent to the desired form. Expansion joints are placed in the parapet wall over the crown of the arch. A cross-

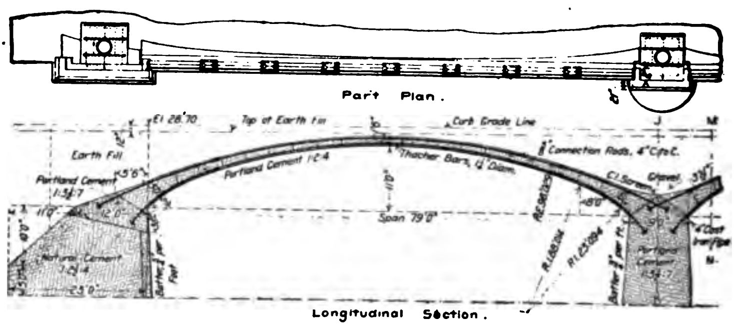


Fig. 601.—Details of 79-Ft. Span. Grand River Bridge, Grand Rapids, Mich.

section of the bridge showing the reinforcement and centering used, is shown in Fig. 600.

Grand River Bridge, Grand Rapids, Mich.—This bridge is a good example of recent reinforced concrete work. It consists of five arch spans, one 87 ft., two 83 ft. and two 79 ft. Figure

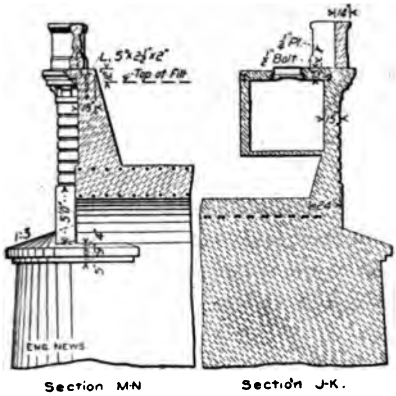


Fig. 602.—Transverse Sections, Grand River Bridge.

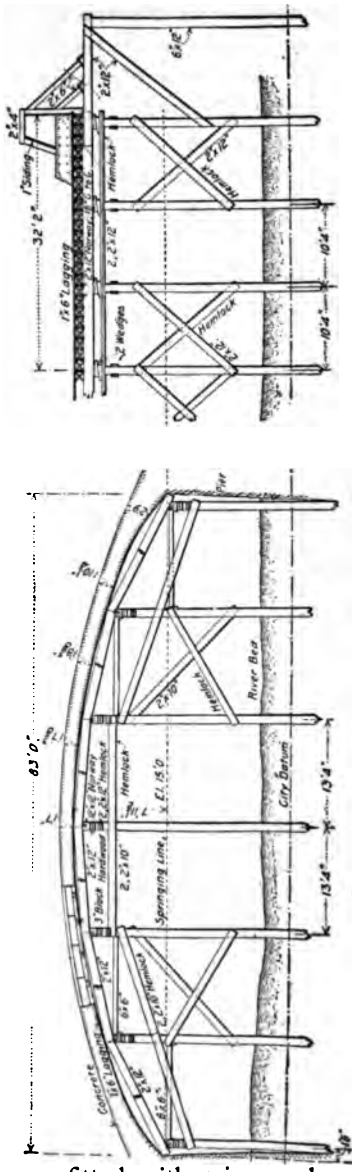


Fig. 603.—Centering for 83-Ft. Span, Grand River Bridge.

601 shows a longitudinal section and part plan of the 79-ft. span. The arch ring is 1 ft. 6 ins. thick at the crown and 3 ft. at the spring. The bridge has solid spandrel walls as shown in Fig. 602. The section of the 83-ft. span is shown in Fig. 603, together with centering used to support the forms. The centering consists of posts or piles driven to a firm bearing and braced together by transverse and longitudinal timber. The reinforcement consists of two lines of 1¼-in. Thacher rods placed 3 ins. from the intradosal and extradosal faces of the arch ring. Each pair of rods is connected every 4 ft. by means of a ⅝-in. rod with a hook at each end. The reinforcing rods are fitted with 3-in. washers and nuts, which give them an anchorage at the abutments, and are made continuous from end to end of span by means of turnbuckles.

The method of constructing the arch rings is as follows: The endmost sections of the reinforcing bars, which had been anchored into the piers and abutments during their construction, were bent down to the curve of the arch ring and connected with the arch rods proper. Scantling placed transversely across the lagging of the center served to block up the soffit rods, while the upper rods were held in place by the connecting rods already mentioned. A wet concrete was deposited and worked in underneath the lower rods. After these were embedded a stiffer concrete was deposited and rammed in 6-in. layers. The arch ring

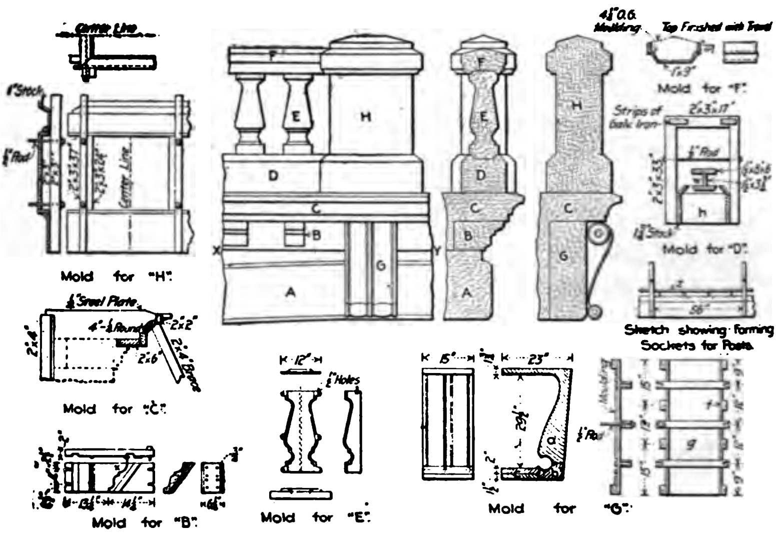


Fig. 604.—Details of Railing and Forms, Grand River Bridge.

was built in transverse sections, and each section was completed in a continuous operation in one day. The crown section was built first and then the two skew back sections, and last the intermediate section; the entire ring being completed in five days. Fig. 603 shows the spandrel wall forms and method of bracing same. Expansion joints in spandrel walls were formed by simply laying the concrete against a vertical form and then butting the concrete of the following section against this smooth surface with a sheet of tar paper inserted between. Fig. 604 shows details of railing and forms used in their construction.

The loadings assumed in the design of this bridge were as follows:

Dead Load.	Lbs. per cu. ft.
Concrete	150
Earth filling	120
Pavement 12 ins. deep	150
Live Load.	**Lbs. per sq. ft.**
Center 20-in. roadway	250
Remainder of roadway	150
Sidewalks	100

A concentrated load was assumed on roadway consisting of a 15-ton steam roller having axles 11 ft. centers with 6 tons on the forward wheel 4 ft. wide and $4\frac{1}{2}$ tons on each of the two rear wheels 20 ins. wide and 5 ft. apart on centers. The ratio of the moduli of elasticity between concrete and steel was taken as 20. The maximum compression allowed on the concrete in the arch ring was 500 lbs. per sq. in., not including temperature stresses, and 750 lbs. per sq. in., including temperature stresses. The maximum tension allowed in the concrete in the arches was, including temperature stresses, due to a variation of 40°, 75 lbs. per sq. in. The maximum shear allowed was 75 lbs. per sq. in. It was required that the ribs reinforcement under a stress not exceeding 18,000 lbs. per sq. in. must be able to take the entire bending moment of the arch without aid from the concrete. It was also required that the area of steel at the crown of the arch should be at least 2% of the total area of the arch at the crown.

The Luten Arch.—A type of arch bridge which has successfully competed with steel truss bridges is the Luten arch. In this bridge the horizontal thrust is taken up by longitudinal ties extending between the abutments underneath the bed of the stream and buried in concrete. The usual heavy abutments which are necessary when the banks of the stream are not ledge rock are thus dispensed with, only enough material being needed at the abutments to enable the arch ring to be connected to the horizontal ties.

A longitudinal and cross section of one of the largest bridges of this type are shown in Fig. 605. This arch was constructed at Yorktown, Ind. The span is 95 ft. and rise of the arch from spring to crown is 11 ft. 1 in., or about one-ninth of the span, the springing being at approximately low water level. The depth of the water at mid-span is about 4 ft. 6 ins., making a total height of opening of 15 ft. 7 ins. The steel tie rods extend from abutment to abutment beneath the bed of the stream and are embedded in a 6-in. concrete pavement. The pavement is provided at both up and down stream edges with aprons projecting downward into the bed of the stream. These prevent

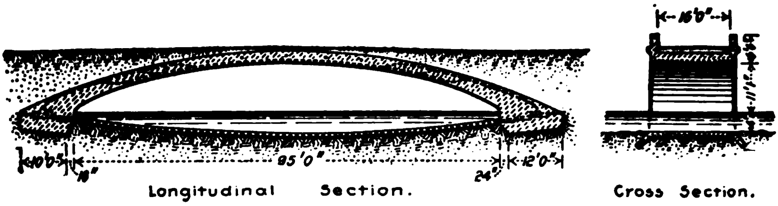


Fig. 605.—Luten Arch Bridge, Yorktown, Md.

the pavement from being undermined, and make the bridge flood-proof.

The reinforcements of the arch rib consist of ¾-in. steel rods, spaced 6 ins. on centers, and arranged in series of single rods

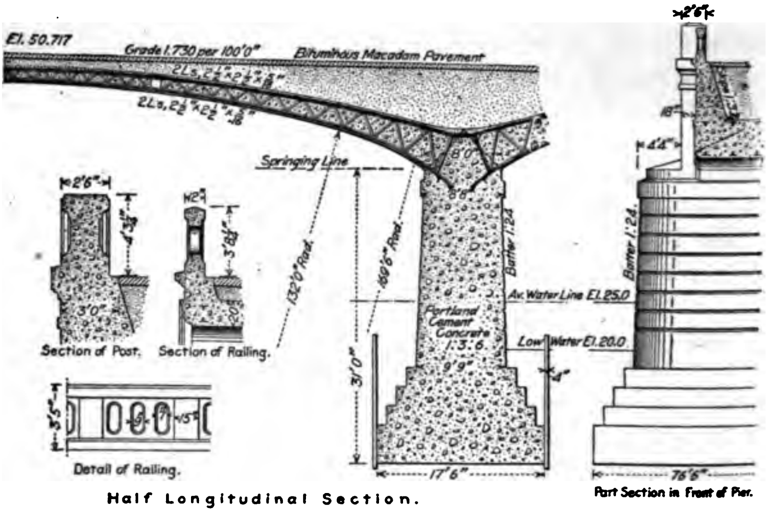


Fig. 606.—Details of Melan Arch Bridge at Dayton, O.

passing through the tension region of the arch rib that is near the intrados at the crown and the extrados at the haunches and abutments.

Since the limits of the tensile regions are not easily determined, the reinforcing rods are arranged to cross the arch rib from intrados to extrados at points distributed over the area of probable minimum moment.

In addition to the ¾-in. steel rods, a reinforcement consisting of electric welded wire netting was attached to the rods throughout the crown of the arch. This netting was of No. 6 × No. 10 wire, spaced 3 ins. and 8 ins. respectively, the heavier wires running longitudinal to the axis of the bridge.

The arch rib was built in parallel rings, the middle ring, 12 ft. wide, being placed first, and the two end rings 3 ft. 6 ins. each, with spandrel and rails following. Each ring was started at both footings simultaneously and was advanced continuously to meet at the crown. The separate rings were bonded together with transverse rods across the roadway, and then afterward into the rails with seven over the crown and two at each abutment. The wire netting was continuous across the entire width of the roadway.

Dayton, Ohio, Melan Arch Bridge.—A seven-span Melan reinforced concrete bridge having a total length of 588 ft. was constructed at Dayton, O., in 1903. The lengths of the spans varied from 69 ft. at the ends to 88 ft. at the center. The width of the bridge in the clear is 54 ft. The proportion of rise of arch to length of span adopted increased from 1-13 to 1-10. The bridge was designed to carry a live load of 150 lbs. per sq. ft. and two lines of 40-ton electric motor cars. The distance between crown of arch and crown of roadway varied from 13 to 15 ins., so that the railway tracks practically rested on the arch ring at the center of the span.

Figure 606 shows a half longitudinal section together with partial cross-sections of the 88 ft. span. The size and arrangement of the reinforcement are shown on the drawing. A 1:2:4 stone concrete was used for the arch ring. A ½-in. coating of cement mortar covered with a coat of tar was used for waterproofing the arch ring. This bridge was designed by the Concrete-Steel Engineering Co. of New York City.

Melan Hinged Arch.—The reinforced concrete arch bridge at Laibach, Austria, is a good example of the application of the Melan system to the construction of a hinged arch bridge. This bridge was designed by Prof. J. Melan, of Prague, and crosses the Laibach River on a skew of 9° 14′. The clear span of the arch is 108.2 ft. and the rise is 14.23 ft. center to center

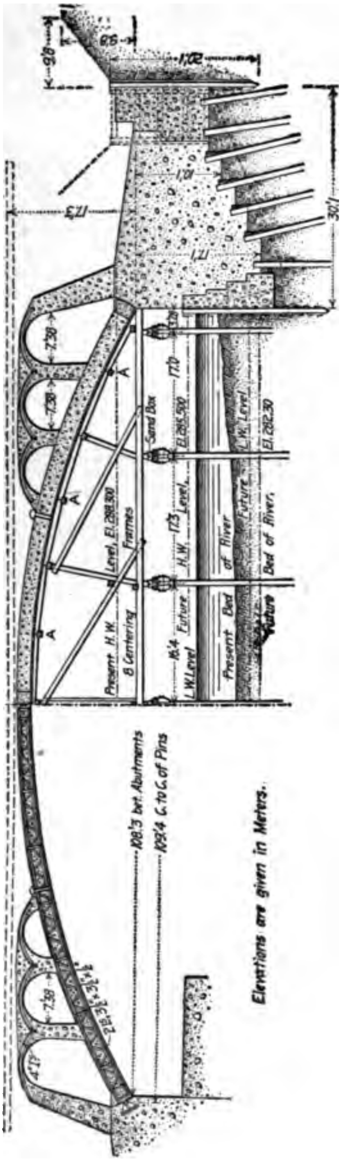
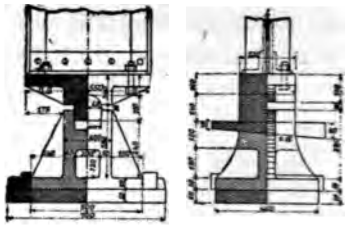
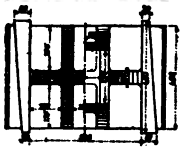


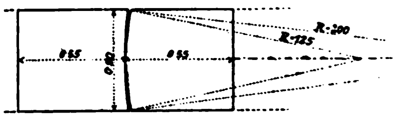
Fig. 607.—Melan Hinged Arch Bridge at Laibach, Austria.



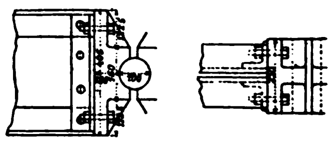


Hinge of Steel Girders.

Fig. 608.—Skewback Hinges, Laibach Bridge.



Concrete Hinge Blocks.



Hinge of Steel Girders.

Fig. 609.—Crown Hinge, Laibach Bridge

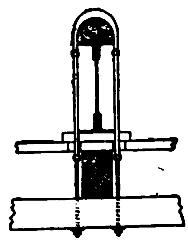


Fig. 610.—Hangers for Centers, Laibach Bridge.

of hinges. Fig. 607 shows arch section, together with arrangement of the reinforcement. The thickness of the main arch ring varies from 20 ins. at the crown to 27½ ins. at the haunches, and 25½ ins. at the skewbacks. The arch reinforcement consists of 14 arched steel lattice girders whose flanges are everywhere about 2 ins. within the concrete and spaced from 3.3 to 3.8 ft. centers. Four sets of steel cross-frames parallel with the axis of the bridge connect and brace these skeleton trusses. Three secondary arches near each end of the main arch assist in carrying the roadway. These secondary arches have spans of 7.4 ft. and their intrados is semi-circular. They have a thickness of 6 ins. in the center, but thicken rapidly toward the haunches. The reinforcement consists of curved 4 in. I-beams spaced 3 ft. and 3 ft. 10 in. centers.

Details of the hinges are shown in Figs. 608 and 609. These hinges are backed by blocks of concrete moulded several months before erection was begun. Full and uniform contact of the curved abutting faces was secured by placing in the joint a strip of hard lead 4 ins. wide and 1-16 in. thick.

The center hinge of the steel ribs is a simple abutting pin joint as will be seen from the drawing, but the skewback hinges of the ribs have wedges for accurate adjustment of the span.

After the arch was completed and the centering removed, the hinges of the metal ribs were encased in concrete.

The character of the falsework is also shown in Fig. 607. Eight centering frames were used, spaced about 7 ft. apart and braced together by diagonal timbers. Each frame rested on 7 supports consisting of single piles. On the top of each support was fixed a sand box. The upper members of the centering frames were not continuous, but were cut in the middle of each panel and here rested on cross timbers hung from the steel reinforcing ribs by means of hangers, as shown in Fig. 610.

The Gruenwald Bridge.—The Gruenwald Bridge, at Munich, is the largest reinforced concrete bridge thus far built. This structure consists of two arched spans of 230 ft., and of five 28-ft. girder approach spans. The arched spans, which are three-hinged arches, are of the type with open spandrel construction supporting the beams and slab roadway. The rise of the arches is 42 ft. center to center of hinges. The roadway is 30 ft. wide over all, with a 16.5 ft. roadway and two 5-ft. sidewalks. The

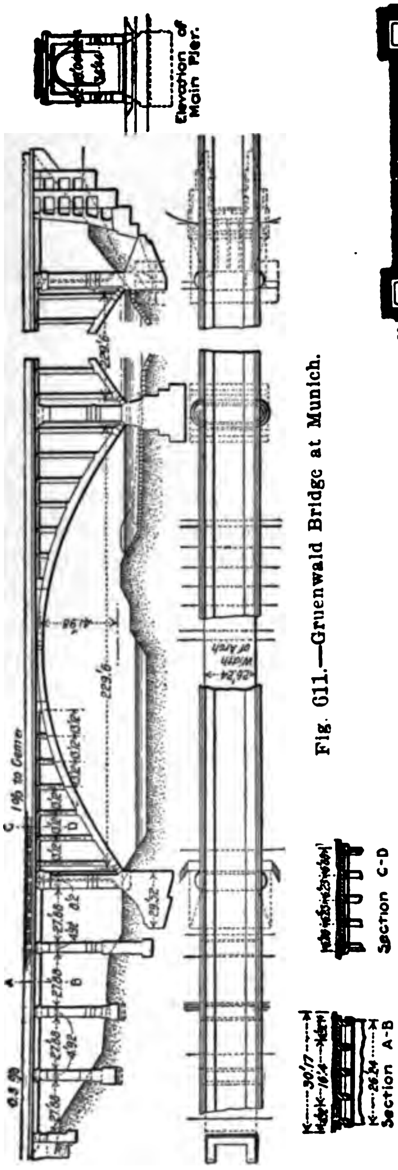


Fig. 611.—Gruenwald Bridge at Munich.

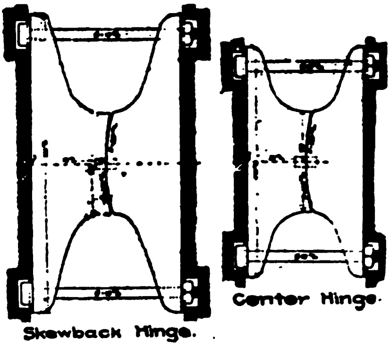
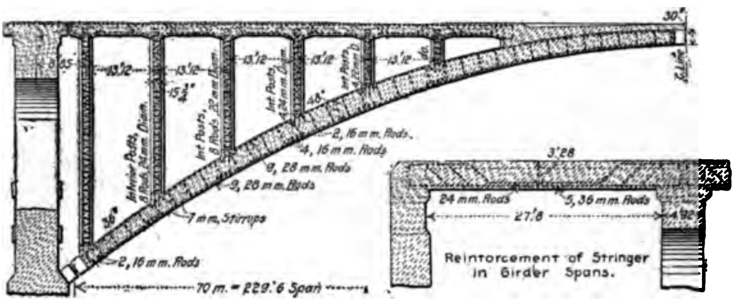


Fig. 612.—Details of Hinges for Gruenwald Bridge.

arches are 26.2 ft. wide, and have a thickness of 30 ins. at the crown, 36 ins. at the springing line, and a maximum thickness of 48 ins. at the quarter points. Figure 611 shows partial elevation, plan and section of the bridge.

The hinges are steel castings with a convex-concave rolling surface. Fig. 612 shows detail of hinges. The convex face has a radius of 8 ins. and the concave face a radius of 10 ins. A dowel at the center provides security against accident. The hinge castings were made in lengths of 31½ ins., and adjoining pairs were spaced 2 ins. apart, giving ten pairs of hinges to each joint. The back face of each casting was planed, and a lead plate about ⅛ in. thick was placed between the castings and arch block against which it abuts. Special blocks of concrete were necessary on account of the excessive bearing back of the lead plates, 1,400 lbs. per sq. in. The blocks were moulded in planed cast-iron moulds, and were provided with transverse reinforcing bars to prevent splitting; these bars extend at right angles to the general direction of the hinge joints, and are distributed uniformly through the thickness of the block. One block was placed under each hinge.

The reinforcement of the arch ring consists of eighteen 28 mm. (1.1 in.) steel rods, nine each at the top and bottom of the ring. The rods were tied together at 3 ft. intervals with ¼ in. stirrups. Round steel rods were used throughout. In addition to this reinforcement, a special reinforcement was placed under each transverse line of posts to aid in the distribution of the post loads on the concrete.



Reinforcement of Arch-Ring and Posts.

Fig. 613.—Main Arch Reinforcement, Gruenwald Bridge.

Two ⅝-in. diam. transverse rods were placed near the top of the arch rib, and four rods of the same size near the bottom of the rib. The arrangement and size of the reinforcing rods are shown in Fig. 613.

The posts supporting the roadway are spaced 3.28 ft. apart transversely, and 6.56 ft. apart longitudinally. All interior posts are approximately 16x16 ins. in section, and vary in height from 5 ft. to 38 ft. Their reinforcement consists of longitudinal rods tied together by ¼-in. rods spaced 14 ins. on centers. The longitudinal reinforcements vary from about eight rods $1\frac{1}{8}$-in. diam., to four rods ⅞-in. diam. The posts in the outer rows were widened out at the outer face to a T-section so as to present a face 28 ins. wide in side elevation. Their reinforcement varies from eight rods $1\frac{3}{8}$-in. diam. to eight rods $\frac{11}{16}$-in. diam. The reinforcing rods of all posts extend downward from 16 to 20 ins. into the concrete of the arch ring.

The roadway consists of a flat plate 7.9 in. thick, and of a 6.56-ft. span resting on five lines of longitudinal stringers of 13.12-ft. span supported by the posts.

Fig. 614 shows cross-section of roadway, also sections of roadway stringer over the arch and section of girder spans. The floor plate is approximately 8 ins. thick, and is reinforced in each 13-ft. panel by twelve top rods, eight of ⅜-in. diam. and four of ½-in. diam., and ten bottom rods, six of ½-in. diam. and four of $^{9}/_{16}$-in. diam. Six of the bottom rods are bent diagonally upwards near the quarter points. The floor stringers over the arches are 10 × 16 ins. in section below floor plate, and are reinforced with four bottom rods, ⅞-in. diam., and two top rods, $^{13}/_{16}$-in. diameter. Over the supporting posts half of the bottom

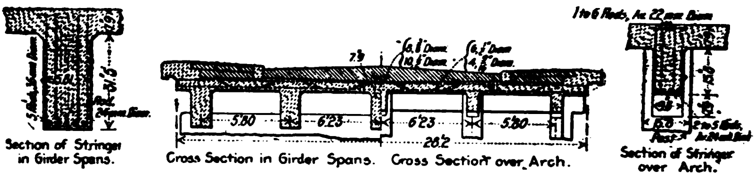


Fig. 614.—Cross-Section of Roadway, Gruenwald Bridge.

rods are bent upward and lie in the top of the beam over the posts. The approach girders are 16 × 32-in. in section below the floor plate, and are reinforced with 1 rod $1\frac{5}{8}$-in. diam. and 5 rods 1⅜-in. diameter. The arrangement of the rods is shown in Fig. 614. This bridge was designed to carry a uniformly distributed live load of 82 lbs. per sq. ft., and a concentrated load consisting of a 22-ton road roller. The maximum compression on the concrete was taken at 510 lbs. per sq. ft. (36 kg. per sq. cm.).

Parabolic Arch Bridge, Wabash, Ind.—Figure 615 shows elevation and cross sections of a parabolic arch bridge with a clear span of 75 ft. The reinforcement used was Kahn bars, which gives an unusual arrangement of the steel in the arch ring. Two spans of 75 ft., together with the approaches, made up a total length of this bridge of 240 ft.

The arches carry solid spandrel walls, with earth filling between. The spandrels were designed as vertical cantilever slabs.

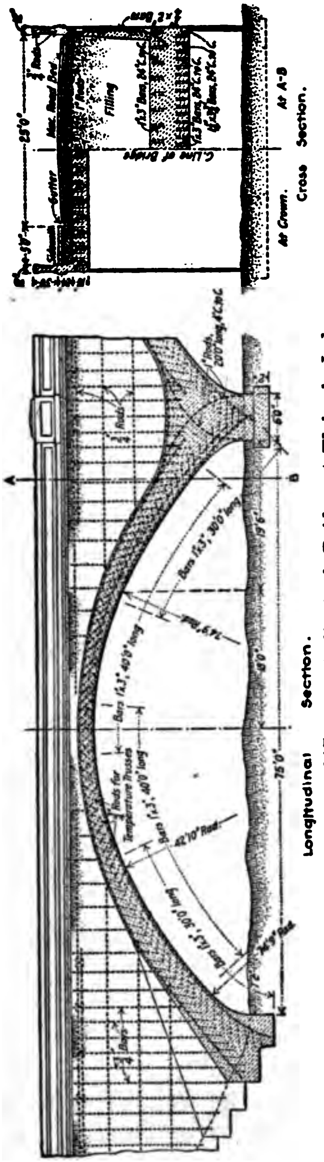


Fig. 615.—Parabolic Arch Bridge at Wabash, Ind.

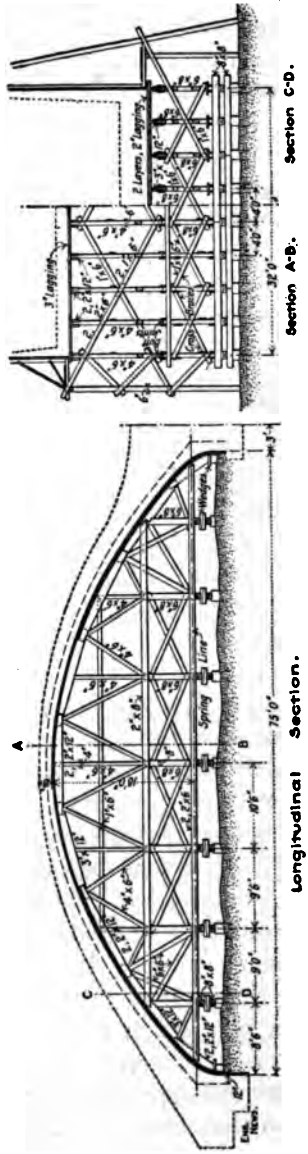


Fig. 616.—Centering for Wabash, Ind., Bridge.

The arch ring is 18 ins. thick at the crown, and 3 ft. 4 ins. at the haunches. The size and arrangement of the reinforcing bars are shown in Fig. 615. Figure 616 shows framing of centering used in the construction of this bridge.

Ribbed Arches.—Ribbed arches have not thus far been used to any extent in this country, although they are popular in Continental Europe. The saving in materials and reduction in dead weight are items which deserve careful consideration in localities adapted to the use of this type of structure. The cost of forms, however, is higher than for the solid arch. The 75-ft. bridge at Grand Rapids, Michigan, and the Deer Park Gorge Bridge, near La Salle, Ill., may be taken as examples of ribbed arch construction of American design.

The Grand Rapids Bridge.—The Grand Rapids Bridge is used as an approach to a steel truss bridge across the Grand River, at Grand Rapids, Mich. This bridge has a span of 75 ft., and consists of seven parallel parabolic arch ribs, supporting a slab and girder floor by means of columns. Five ribs 2 ft. wide, 50 ins. deep at springing lines, and 32 ins. deep at the crown, support the 21-ft. roadway, while the sidewalks are supported by two ribs, each 1 ft. wide, 50 ins. deep at springing and 25 ins. deep at the crown. The soffits are all shaped to the same curve of three centers, approximating very closely a parabola of 14 ft. rise and 75-ft. span. The linear arch of the main ribs has a rise of 12.7 ft. The ribs are connected and braced by 4-in. reinforced concrete slab webs lying near the neutral axis of the ribs. These were deemed necessary to stiffen the ribs to resist the impact of floating ice and drift during floods. The webs extend between the main ribs from each abutment to the second row of columns, and between the outer ribs and those adjacent they extend to the third row of columns. The general arrangement of ribs, struts and posts are shown in plan, elevation and section in Figs. 617, 618 and 619. As will be seen, in addition to the webs, the ribs are further stiffened by 8 × 8-in. concrete struts at each of the middle columns. The longer columns are also connected by 4-in. vertical webs of reinforced concrete. The columns under the roadway are 12 × 12 ins., with a 1-in. chamfer on the corners. The sidewalk columns are 10 × 12 ins. The sidewalk slabs are 9 ft. wide, and overhang the 6 × 12-in. longitudinal beam 3 ft. They are reinforced with ½-in. bars spaced 8 ins. centers. The roadway, which is built in the form of a trough, has an 8-in. reinforced

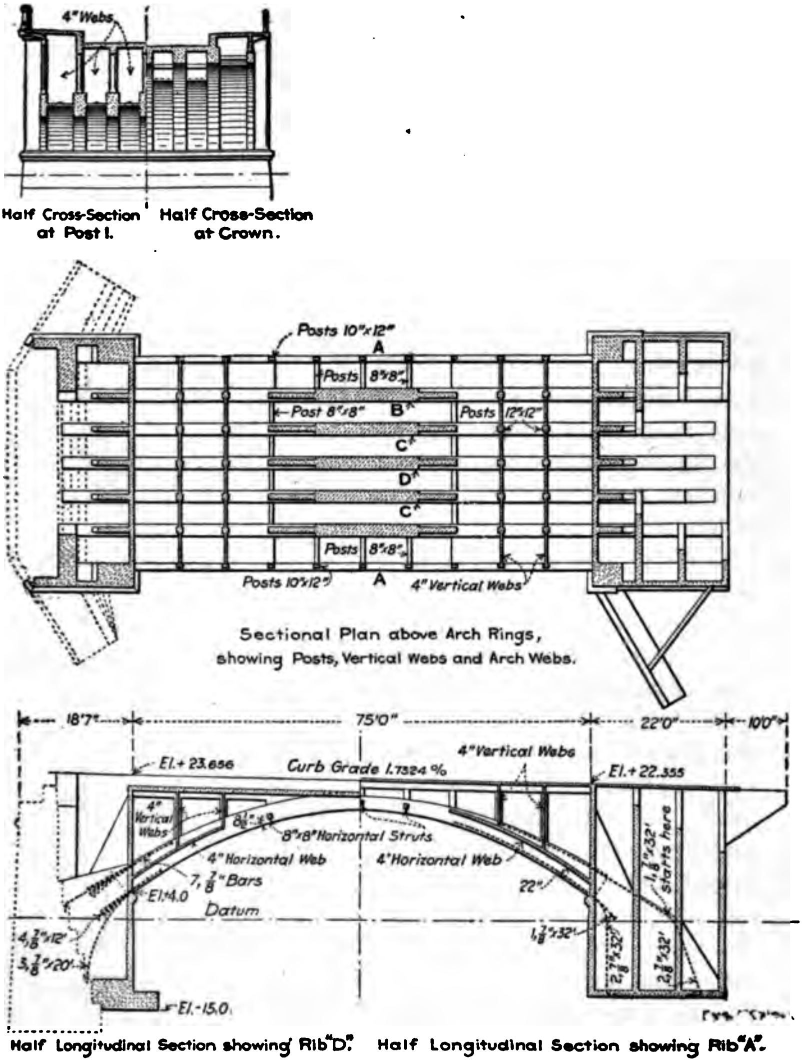


Fig. 617.—Plan and Sections of Grand Rapids, Mich., Bridge.

slab. Sections of arch rings, cantilevered sidewalk, vertical posts and struts are shown in Fig. 618. The arrangement of the shear bars in the arch ribs is shown in Fig. 619. This figure also shows

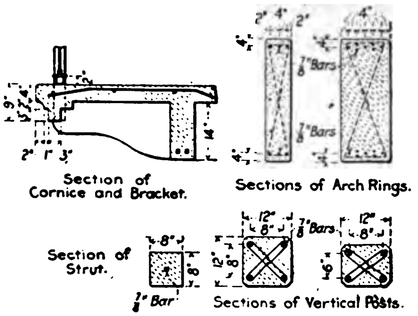


Fig. 618.—Details of Arch Rings, Spandrel Posts, etc., Grand Rapids Bridge.

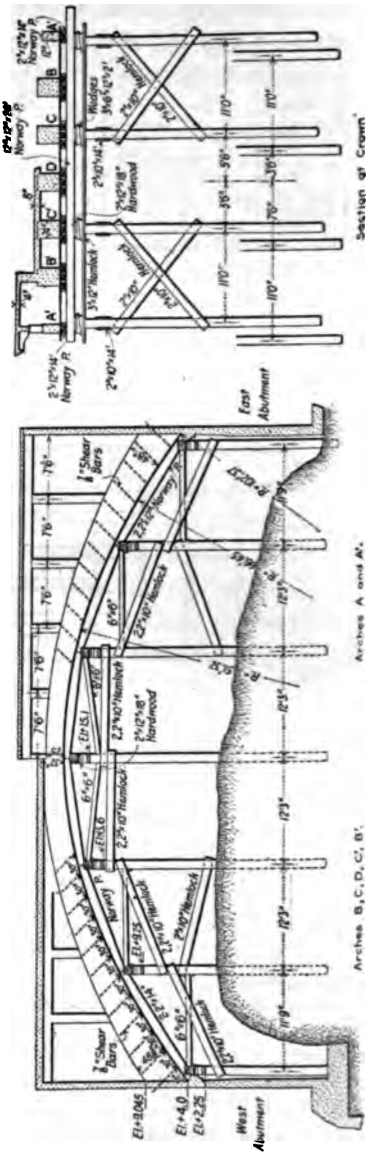


Fig. 619.—Centers for Grand Rapids Bridge.

construction of centering used for this bridge. As will be seen, it consists of vertical posts braced together to form bents supporting the timbers carrying the lagging, which was only of sufficient length to form bottom of box for the rib.

The arch ribs were built complete in one operation, usually two ribs being placed in one day. The top of the crowns was roughened to form a bond with the roadway slab and girders into which the main ribs merged. The reinforcement for the columns was placed when the rib concrete was placed and extended nearly to the bottom of the ribs. After the ribs had been concreted, the forms for the post girders and for slabs were put in position, and the concreting completed in one operation.

This bridge was constructed without hinges, but expansion joints of tarred felt and tar were made across the bridge at the center and two abutments, the center joint extending through the slabs to the arch ribs and columns.

This bridge was designed to carry a 15-ton road roller, or a 24-ton electric car or a uniform live load of 250 lbs. per sq. ft. on roadway and 100 lbs. per sq. ft. on sidewalks. A factor of safety of two for dead loads and four for live loads was used.

It is stated that the cost of forms and concrete in arches, slabs, sidewalks and roadway, etc., 290 cu. yds. being used, was:

		Cost per cu. yd. of concrete.	
Forms:	Material	$3.70	
	Labor	3.03	
			$6.73
Concrete:	Materials	$3.22	
	Labor	3.57	
			6.79
Total			$13.52

Deer Park Gorge Bridge.—This bridge may be taken as an example of light construction, as it is intended only for a foot

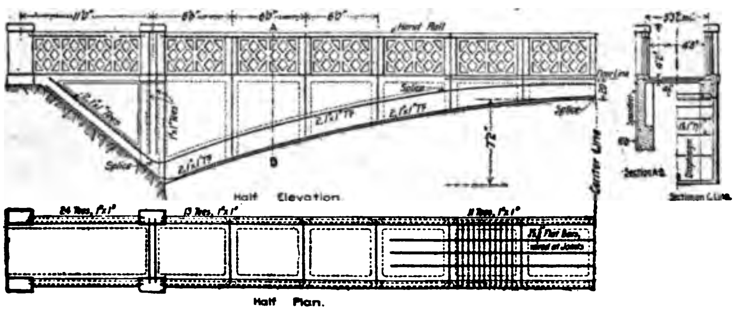


Fig. 620.—Deer Park Gorge Bridge.

bridge. The clear span is 72 ft., length over all 95 ft., and distance between hand railings 4 ft. 6 ins. The rise of the arch is 7 ft. 6 ins., with a camber of 4 ins. Figure 620 shows half plan elevation and cross-section of the bridge. The arch rings are 16 ins. in thickness, and vary in depth from 20 ins. at the crown to 24 ins. at the haunches. From the arch ring to the floor line there is a 7-in. spandrel wall, on which is carried the 4-in. horizontal floor slab. A series of 6-in. diaphragms spaced 5-ft. centers, extending from the floor line to the bottom of the arch ring, were placed between the ribs.

Each rib is reinforced with four 1 × 1-in. × 0.87-lb. steel T-bars, located as shown in Fig. 620. The bars are in four lengths, and have wired splices staggered, and are carried entirely through the abutment ends of the ribs. The diaphragms are reinforced with transverse T-bars, as shown. Vertical T-bars are run up from the ribs into the railing posts. The floor slab is reinforced with 1 × 1-in. × 0.87-lb. T-bars running transversely, and spaced

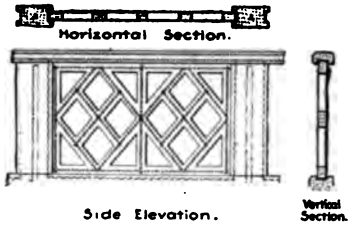


Fig. 621.—Details of Hand Railings, Deer Park Gorge Bridge.

about 6 ins. apart. Three 1 × ⅛-in. flats are located in the floor slab, and run longitudinally the full floor length.

Figure 621 shows details of reinforced concrete hand railing.

Figure 622 shows side elevation and section of centers and forms used for this bridge. The size of timbers and method in which they are placed are shown in the figure. On the top of the

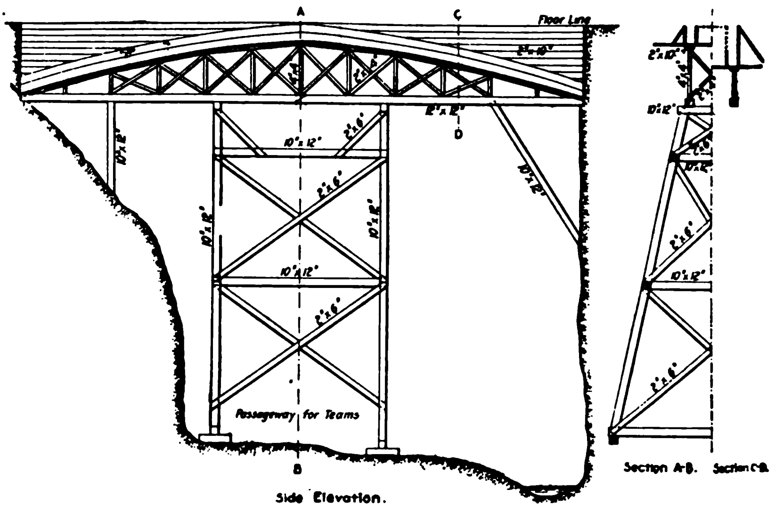


Fig. 622.—Centers for Deer Park Gorge Bridge.

12 × 12-in. longitudinal cross timbers, which are supported as shown, were erected 2 × 4-in. vertical bracing to support the lagging. The lagging consisted of 2 × 10-in. plank placed lengthwise and flat, with staggered joints, and on these a floor of 2 × 10-in. transverse planks were placed, and acted as a working

floor as well as bottom for the forms. The ribs were concreted first, then the forms for the remainder of the structure put in position, and the concreting completed. A 1:2:4 concrete was used. A 1-in. mortar finish was applied to the floor for a wearing surface. It is claimed by Mr. J. B. Strauss, who designed this bridge, that a saving of from 25 to 35 per cent. resulted from the use of the ribbed arch type.

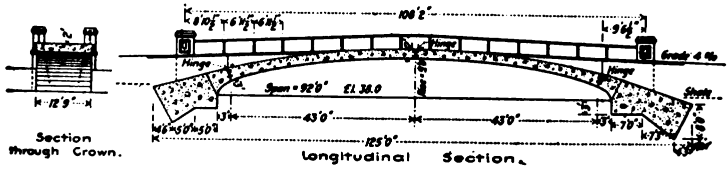


Fig. 623.—Three-Hinged Arch, Brookside Park, Cleveland, O.

Three-Hinged Concrete Arch Bridge, Brookside Park, Cleveland, O.—This concrete bridge over Big Creek, in Brookside Park, Cleveland, O., is one of the few three-hinged arch bridges constructed in this country. The shape of the arch is that of a semi-ellipse, whose major axis is 92 ft. and semi-minor axis 9 ft. The arch proper, however, stops at the abutment hinges, giving a span length between abutment hinges of 86 ft. 4½ ins. The rise of the arch, i. e., the vertical distance between center and abutment hinges, is 5 ft. 2½ ins., making an

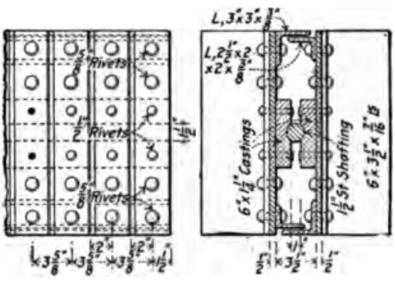


Fig. 624.—Hinges for Brookside Park Bridge.

extremely flat arch. The width of the arch is 12 ft. 9 ins. Figure 623 shows cross and longitudinal sections of this bridge, while Fig. 624 shows details of a part of one of the hinges. The thickness of the concrete at the crown is 2 ft., and at the abutment hinges 3 ft. The abutments rest upon and are buried in firm shale rock. The concrete is entirely unreinforced, with the exception of a few rods used to tie the spandrel walls to the arch

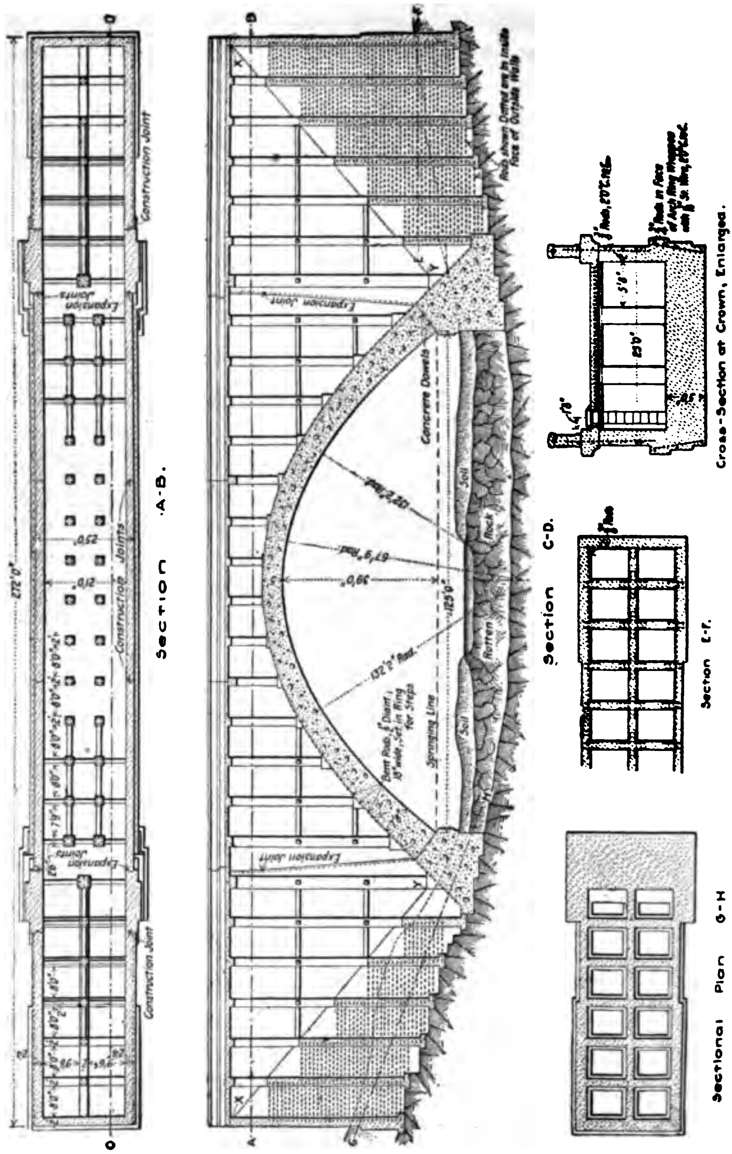


Fig. 625.—Parabolic Arch Bridge Over Piney Creek, Washington, D. C.

proper to prevent any possible failure when the earth filling should be placed on the bridge.

A 1 : 2½ : 5 concrete mixture was used, but the outside faces were made of a 2-in. coating of very rich concrete, which was put in at the same time as the body of the concrete. The arch was further waterproofed with a very heavy coating of asphalt gum.

The position of the hinges was so chosen as to prevent any

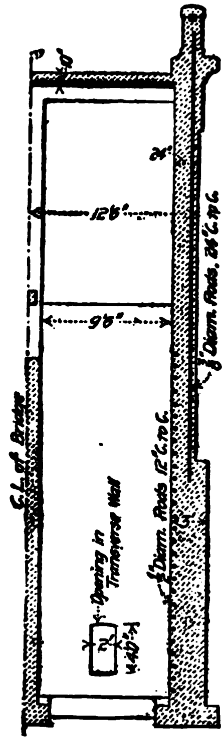


Fig. 626.—Spandrel Wall, Piney Creek Bridge.

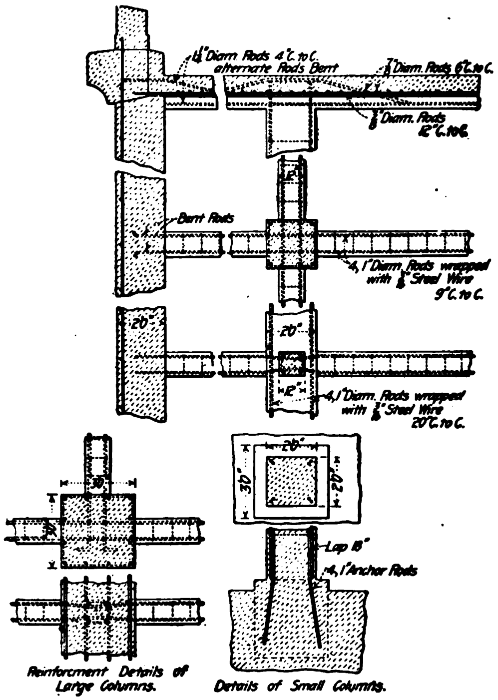


Fig. 627.—Interior Spandrel Construction, Piney Creek Bridge.

tension on the concrete. The hinges were built up of plates and angles, and have cast-iron bearing plates carefully fitted to 2½-in. diam. steel shafting, so as to secure a uniform bearing on the shaft. Before being placed in position the hinges were thoroughly greased and the joints all carefully protected against rust. At the joints the concrete was separated ½ in., and this opening carefully closed by calking it with pure asphalt. The maximum compressive stress coming upon the concrete is 500 lbs. per square inch. This bridge was designed by A. W. Zesign, Asst. Park Engr., Cleveland, O.

Piney Creek Bridge, Washington, D. C.—The parabolic concrete bridge over Piney Creek at 16th Street, Washington, D. C., is an unusual type of concrete bridge. The arch ring has the curve of a parabola, a clear span of 125 ft., and a rise of 39 ft. The spandrels are hollow, and a framework of reinforced concrete between curtain walls carries the floor slab. The spandrels and the floor slab are of reinforced concrete, but the arch ring

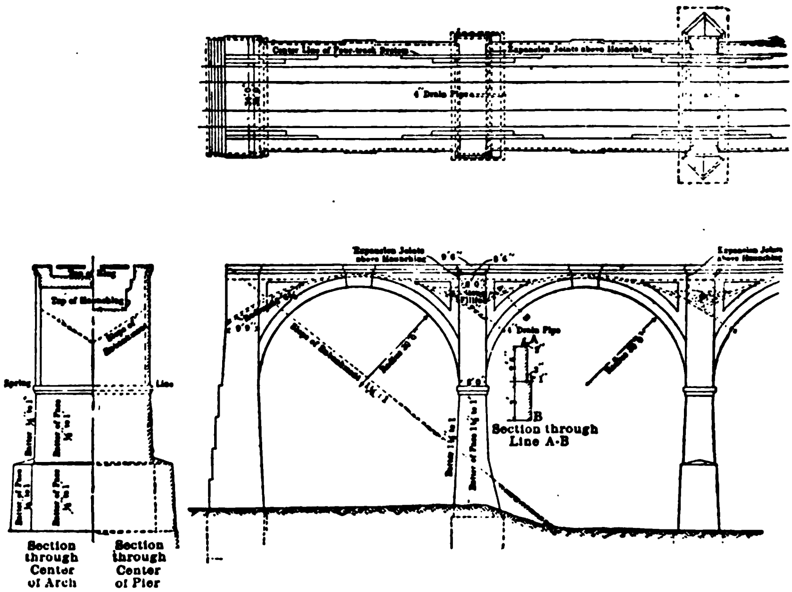


Fig. 628.—Pennypack Creek Bridge, New York Short Line Railway.

is unreinforced, except for a net of face bars to resist surface cracking. Figure 625 shows horizontal and vertical sections of the bridge. The spandrel constructions consist of two solid spandrel walls, and between them two rows of columns, all supporting a ribbed floor slab. The longer spandrel columns are stiffened laterally by horizontal braces between columns in both directions; the transverse braces running with the spandrel walls. Columns, walls and girders are all reinforced, as shown in detail in Figs. 625-627. The spandrel construction is continued over the abutments, with certain modifications. First one row of columns is used instead of two, and the lower portions of these columns are connected by walls to form closed cells, which are filled with earth.

The reinforcement of the floor slab is indicated in Fig. 627. The slab is stiffened by transverse ribs over the columns. These ribs or girders vary in section from 24 × 16 to 36 × 18 ins., and are reinforced with from six to nine 1¼-in. rods spaced 4-ins. centers. Alternate rods are bent up at the ends of span, and carried over the columns. The slab reinforcement consists of ⅜-in. transverse rods spaced 6-ins. centers and ⅞-in. longitudinal rods spaced 12-ins. centers.

Pennypack Bridge.—An unreinforced concrete bridge 348 ft. long, having an extreme height of about 80 ft., was recently con-

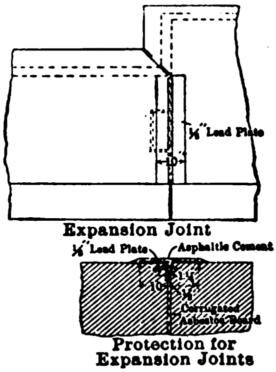


Fig. 629.—Expansion Joint, Pennypack Creek Bridge.

structed for the New York Short Line R. R., over the Pennypack Creek, about 12 miles from Philadelphia. This bridge is a monolithic structure, and comprises four full centered arches of 60-ft. span, supported by two abutments and four piers, all of concrete. Figure 628 shows plan, elevation and sections of two end spans. The expansion joint at piers is also shown in detail. The arches have a radial thickness of 3 ft. at the crown, increasing over the haunches to a maximum thickness of 11 ft. on the center line of the piers, where the upper surfaces of two arches meet. The upper faces of the arch terminate on the plane of the spandrel walls, which, like the piers, are carried up to the level of the base of rail.

Vertical expansion joints are provided in the spandrel walls over each pier, as shown by Fig. 629.

Figure 630 shows longitudinal section through arch and abutment, while Fig. 631 shows section at crown. Details of centering and forms for piers are also shown in these figures. Sizes of materials used are marked on drawing.

A 1:2:4 concrete was used for all parts of the bridge above the springing line, while a 1:3:6 concrete was used exclusively below these points.

Weak Haunch Reinforcement.—The use of extremely flat arches necessary to secure sufficient head room or waterway has led to the use of three or five centered or elliptical arches. In many cases where such arches have been used, the arches have cracked on the haunches, the cracks opening up on the extrados; and where a solid parapet wall and railing of monolithic construction has also been employed, the cracks extend through these. These cracks, while not endangering the safety of the bridge, are both unsightly and objectionable. The flat portion of the arch undoubtedly acts as a beam, and it would appear to be desirable in some cases to design the bridge as a cantilever or fixed beam, or at least give it a segmental form, and increase the extradosal reinforcement considerably beyond what is theoretically required. The Seeley Street Bridge, Brooklyn, N. Y., shows bad cracks through the haunches, and reaching through the spandrel walls. A low unit stress in both steel and concrete should also be used, say, from 9,000 to 12,000 lbs. per sq. in. for the former and 250 to 350 lbs. for the latter. When possible, it will be well to remove the centering and allow the arch to settle in place before building the parapet walls and railing. In many cases it will be found desirable to mould the railing in sections, and set it up after the arch ring and parapet have been completed.

Expansion Joints.—Expansion joints are usually provided for the spandrel walls or arches at each pier and at the abutments, to allow for some movement due to temperature changes. Expansion joints may be made by placing in the joint several thicknesses of corrugated asbestos board, protected by a ⅛-in. lead plate folded into the joint to form a trough at the top. An asphaltic coating may then be placed over the lead, covering the whole; this gives an elastic and perfectly waterproof joint. Tongued and grooved joints are sometimes used, but in general a lead covering such as that shown in Fig. 632, it is believed, will prove the most satisfactory. The joints shown in the

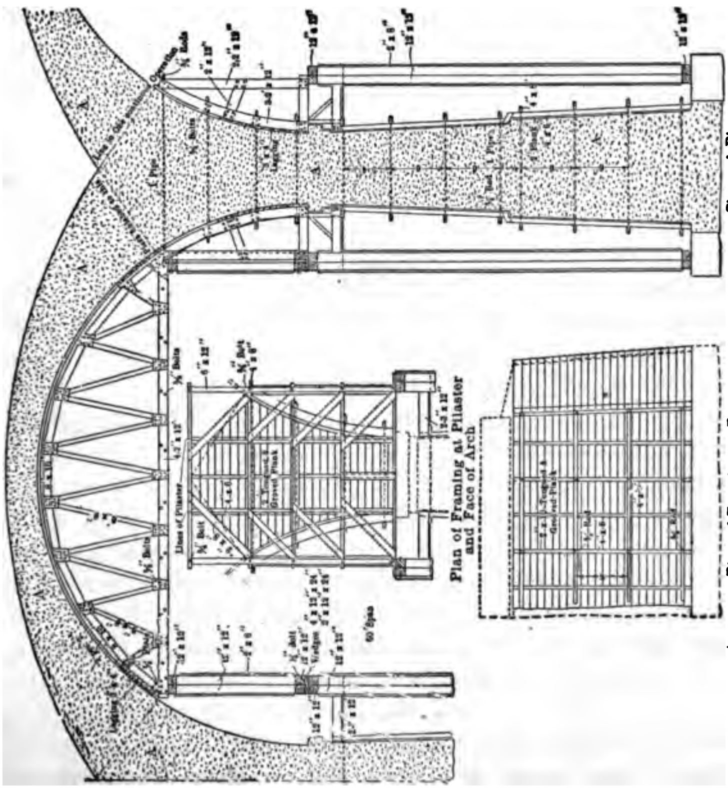


Fig. 630.—Details of Arch and Pier, Pennypack Creek Bridge.

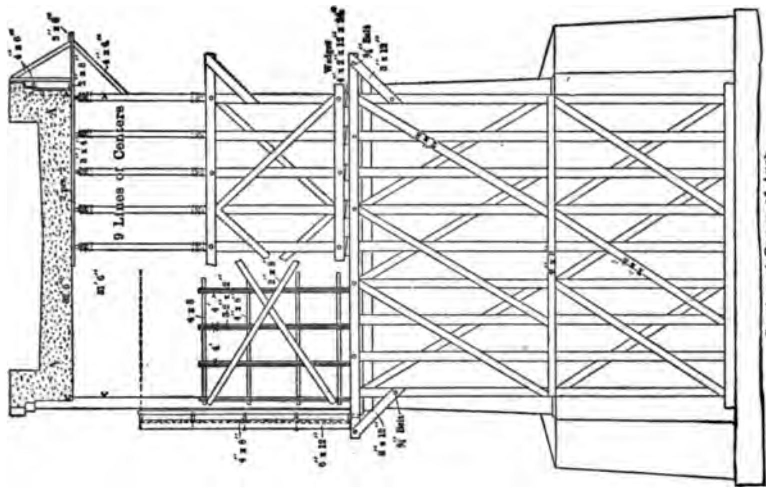


Fig. 631.—Transverse Section, Pennypack Creek Bridge.

figure were used on the Big Muddy Bridge, Illinois Central Ry. See also Fig. 629 for expansion joint used for the Pennypack Bridge.

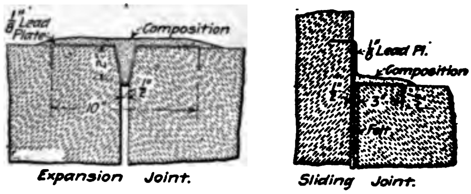


Fig. 632.—Expansion Joints, Big Muddy Bridge, Illinois Central R.R.

Waterproofing and Drainage.—It is customary to waterproof the top of the arch ring and arches over the spandrels either with a Portland cement grout or a coating of asphalt. The methods for waterproofing described in another chapter may be applied to bridges. Pipe drains should also be provided to carry off the water and discharge it at points where it can do no harm. It is desirable where the expense may be allowed to use bronze for the drain pipes, as iron or steel pipes will rust and discolor the masonry.

CULVERTS.

Culverts of arch form are built both in plain and reinforced concrete, while reinforced concrete is extensively used for the construction of box culverts. For small-sized culverts reinforced concrete pipes may be used to replace cast iron and vitrified clay pipes, in many cases with considerable saving in cost. The smooth waterway obtained when concrete is used in the construction of culverts increases considerably their capacity, and prevents stoppage of the waterway by drift, which is a common trouble met with in the ordinary loose stone culverts often used in high embankments. Again, when such construction is used with suitable wing walls, no trouble may be feared from the water undermining the earth embankment and dropping the track during time of flood. Reinforced concrete culverts have been adopted as the standard construction by a number of American railroads.

Figures 633 and 634 show the standard pattern used by the Chicago, Burlington & Quincy Ry. for standard box culverts, the first for clear spans up to 7 ft., and the second for spans of

8 ft. and over; while Fig. 635 shows the standard pattern for arch culverts of small span. The dimensions L and l in the box culvert designs are determined by the formulas:

$$L = \frac{10}{3}h + x + 3 \text{ ft., and}$$

$$l = \frac{10}{3}h + x,$$

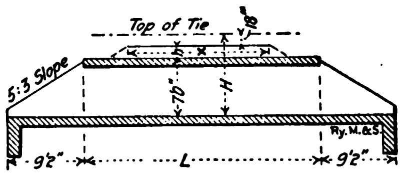


Fig. 633.—Standard Box Culvert for Clear Widths of 7 Ft., C., B. & Q. Ry.

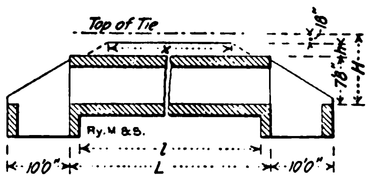


Fig. 634.—Standard Box Culverts for Clear Widths of 8 Ft. and Over, C., B. & Q. Ry.

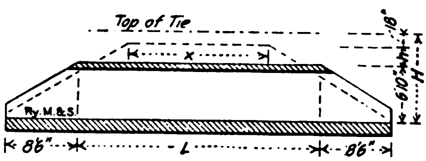


Fig. 635.—Standard Arch Culverts for Interior Dimensions of 4 x 4 Ft.; 5 x 5 Ft. and 6 x 6 Ft., C., B. & Q. Ry.

in which x = width of the roadbed at the crown, and h = height of fill above the culvert. In the arch culvert pattern the dimension L is determined by the formula:

$$L = \frac{10}{3}h + x + 4 \text{ ft.}$$

All other dimensions are determined by the cross-sectional sizes of the waterway.

Table LXXIII., taken from "Railway Maintenance and Structures" for September, 1906, gives the dimensions and thicknesses of roof, floor and side walls, together with amount of concrete used for the various sizes adopted as standards by this railroad:

TABLE LXXIII.

Box Culverts Pattern Fig. 633.

Inside dimensions in feet.	Length of wing walls, ft. ins.	Cu. yds. concrete wing walls.	Cu. yds. conc. lin. ft., barrel.	Thickness side walls, inches.	Thickness roof slab, inches.	Thickness floor slab, inches.
4 × 4	5 10	7.4	0.75	12	12	12
4 × 5	7 6	9.2	0.83	12	12	12
4 × 6	9 2	11.6	0.9	12	12	12
5 × 4	6 1	9.0	0.91	12	14	14
5 × 5	7 9	11.3	0.99	12	14	14
5 × 6	9 6	13.9	1.06	12	14	14
6 × 5	8 0	13.5	1.18	12	16	16
6 × 6	8 0	16.5	1.25	12	16	16
6 × 8	12 9	18.3	1.60	15	16	16
7 × 5	8 4	15.65	1.39	12	18	18
7 × 7	11 5	24.9	1.72	15	18	18
7 × 8	13 0	29.13	1.82	15	18	18
Box Culverts, Pattern Fig. 634.						
8 × 6	10 0	31.0	1.89	15	20	20
8 × 8	13 4	39.7	2.08	15	20	20
8 × 10	10 5	57.1	2.51	18	20	20
10 × 10	17 0	62.3	3.07	18	24	24
10 × 12	20 4	76.0	3.3	18	24	24

The arrangement of the reinforcement and general details of the 4 × 6-ft. culvert are shown in Fig. 636. The general arrangement for the larger sizes are similar in all respects.

Figure 637 shows details of a 10 × 12-ft. box culvert. The structural features of the standard arch pattern of a 6 × 6-ft. culvert are shown in Fig. 638. The dimensions, quantity of concrete and metal used are shown in Table LXXIV.

TABLE LXXIV.

Inside dimensions in feet.	Length of wing walls, ft. ins.	Cu. yds. concrete in wing walls.	Lbs. of metal wing walls.	Cu. yds. concrete per lin. ft., barrel.	Lbs. of metal per lin. ft., barrel.
4 × 4	5 3	6	236	0.5	54
5 × 5	6 11	10	401.7	0.71	76.7
6 × 6	8 6	12	553.5	1.00	103.4

The concrete used for the culverts constructed by the C., B. & Q. Ry. was a 1:3:6 mixture, both gravel and broken stone being used.

Details of a box culvert of 4-ft. span, used by the Great Northern, is shown in Fig. 639. The minimum thickness of concrete used for slabs and walls is 9 ins. The arrangement of the reinforcement is clearly shown in the figure.

A culvert of similar pattern is used by the K. C., M. & O. Ry.

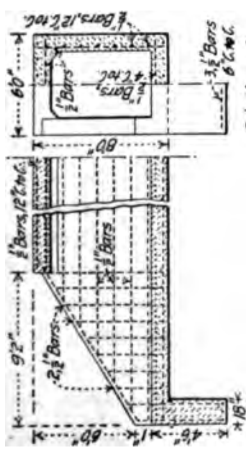


Fig. 636.—Structural Details of Standard, 4 x 6 Ft., Box Culvert.

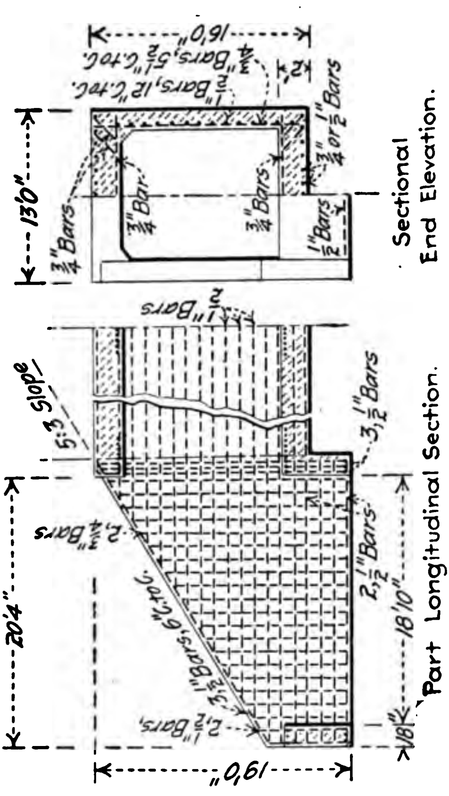


Fig. 637.—Structural Details of Standard, 10 x 12 Ft., Box Culvert.

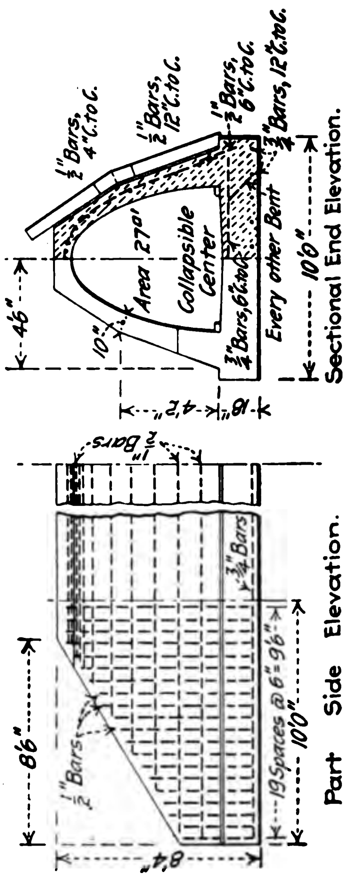


Fig. 638.—Structural Details of Standard, 6 x 6 Ft., Arch Culvert.

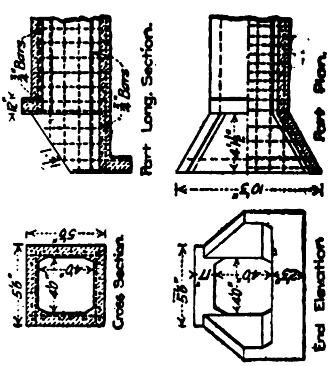


Fig. 639.—Box Culvert for Great Northern Ry.

Figure 640 shows general pattern of a 4 × 6-ft. double box culvert used by this railroad.

Figure 641 shows details of a 20-ft. flat top culvert used by the Cleveland, Cincinnati, Chicago & St. Louis Ry. The details of construction are clearly shown in the figure. It may be noted that several different classes of concrete are used. Class A is a 1 : 4 : 8 concrete; Class B, 1 part cement to 9½ parts gravel; Class C was a 1 : 3 : 6 mixture; Class D, 1 part cement

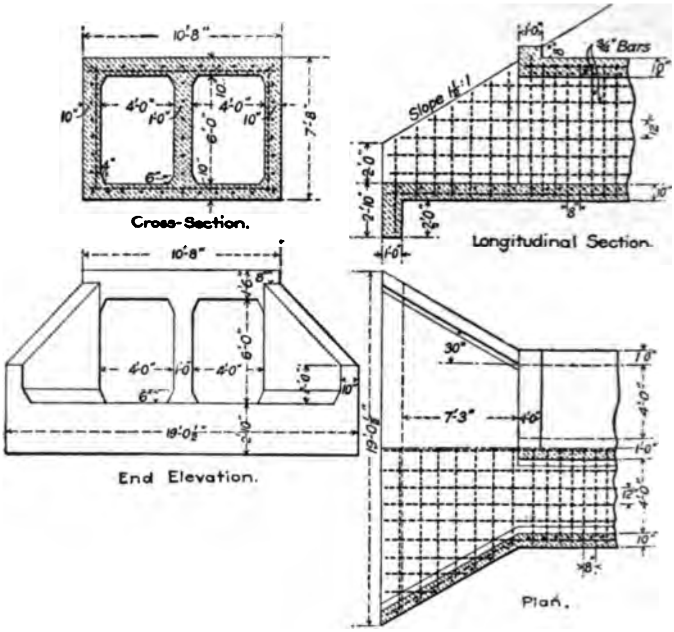


Fig. 640.—Double Box Culvert, K. C., M. & O. Ry.

to 6½ gravel, and Class E consisted of a 1 : 2 : 4 mixture. All of these mixtures are used by the C., C., C. & St. Louis Ry.

Any one of the above standard flat top culverts can be used for double, triple or multiple-span culverts where an increased waterway is needed without the additional expense necessary for the construction of a single span opening. This construction resembles in many respects the old type used for stone box culverts.

Figure 642 shows cross-sections of an unreinforced arch culvert recently built by the Nashville, Chattanooga & St. Louis Ry. This section was used through an embankment from 76 ft. in

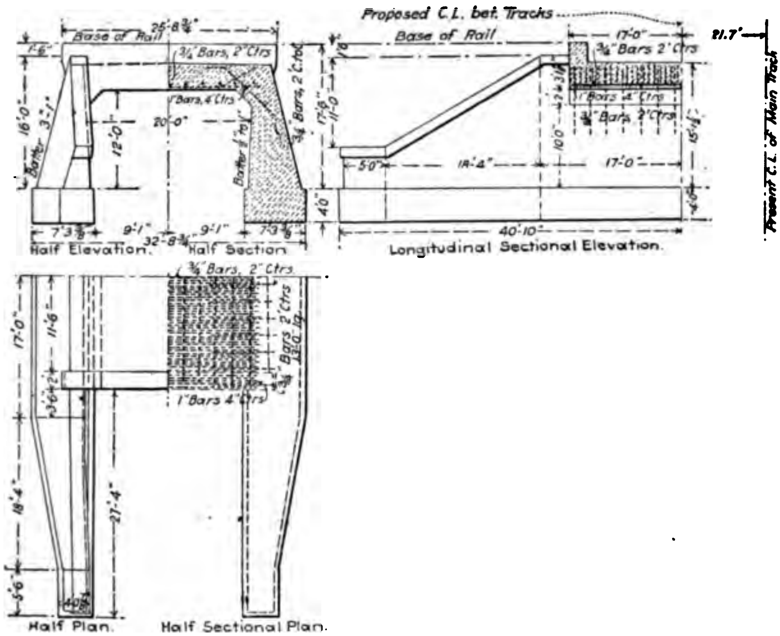


Fig. 641.—Flat Top Culvert, C., C., C. & St. L. Ry.

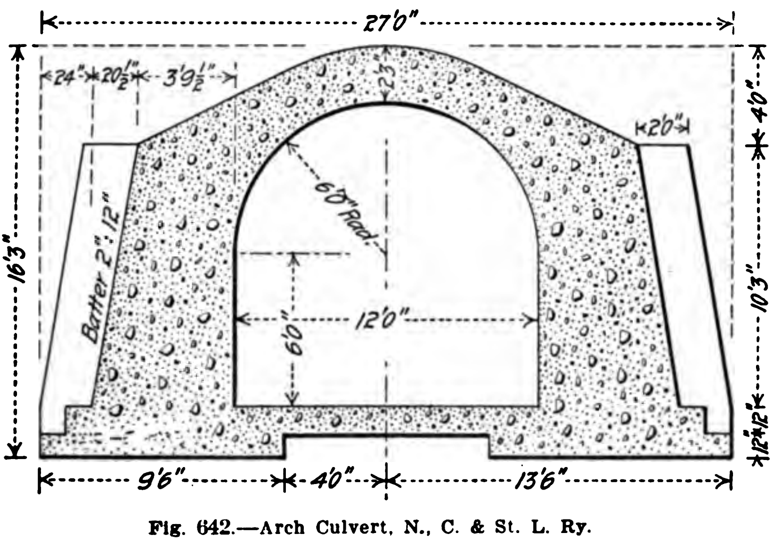


Fig. 642.—Arch Culvert, N., C. & St. L. Ry.

height, and when completed had a length of about 200 ft. The culvert was built in three sections, an intermediate one of 70 ft. and two end ones of about 65 ft. each. The thickness of the

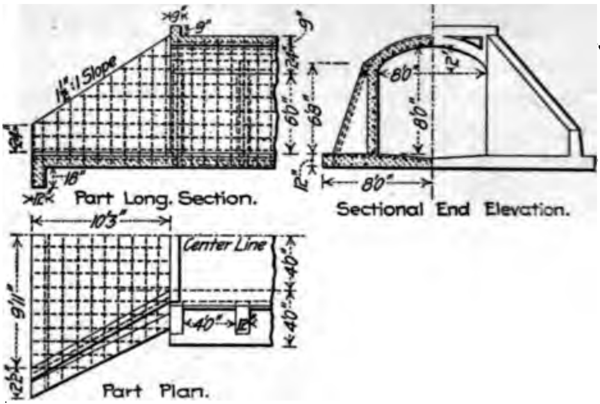


Fig. 643.—Arch Culvert, Great Northern Ry.

crown shown in the figure was for the middle section. This thickness was reduced to 1 ft. 9 ins. for the side sections, where the earth fill brought less load upon the arch. A 1:3:6 slag and concrete aggregate was used.

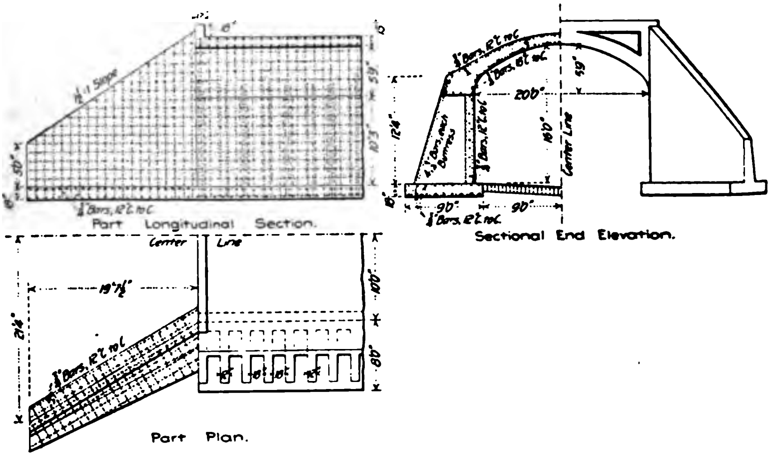


Fig. 644.—Arch Culvert, Great Northern Ry.

A design by Mr. C. F. Graff for an arch culvert of small span used by the Great Northern Ry. is shown in Fig. 643.

Figure 644 shows design for a 20-ft. arch culvert used by the same railroad. The size and arrangement of the reinforcement, together with the general features of the design, are clearly shown in the drawings. In the design of this culvert a height of bank varying from 22 to 40 ft. was assumed. The weight of earth fill was taken at 100 lbs. per cu. ft. A uniform live load of 10,000 lbs. per lineal foot of track was assumed, 50 per cent. added for impact, and a factor of safety of 4 was used on such live load plus impact, and of 2 for dead load. The ultimate strength of concrete in tension, compression and shear was taken at 200, 2,000 and 400 lbs. per sq. in., respectively. Corrugated bar reinforcement was used, with an elastic limit of 50,000 lbs. per sq. in. The weight of the concrete was taken at 150 lbs. per cu. ft. It was found that the weight of the 40-ft. bank increased

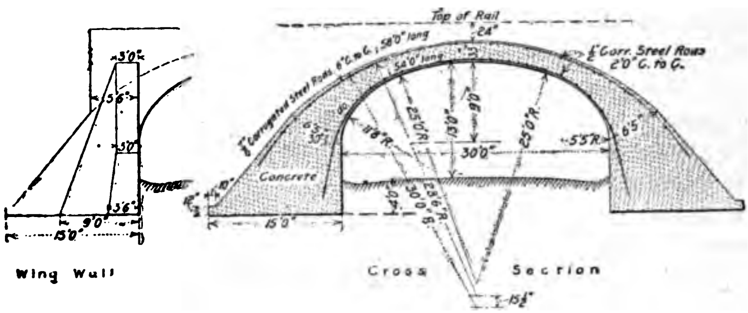


Fig. 645.—Arch Culvert, L. S. & M. S. Ry.

but slightly the section needed to carry a 20-ft. embankment, and this section was used for all heights of embankment up to 40 ft.

The method of using I-beams and old rails as reinforcement for flat top culverts and bridges is described on page 756.

A large culvert of the Monier type is shown in Fig. 645, which gives a part elevation and section of the reinforced concrete arch culvert at Elkhart, Ind., used in the yards of the Lake Shore & Michigan Southern Ry. These culverts, 40, 60 and 160 ft. long, respectively, were built under the main tracks and two groups of yard tracks. The arches have a clear span of 30 ft. and 13-ft. clear headway. The general details are given on the drawing. The Monier reinforcement consists of two lines of ½-in. longitudinal corrugated bars spaced 6-in. centers, and located 2½ ins. from the intradosal and extradosal faces respectively. Across these are placed transverse rods spaced 2-ft. centers. A 1:3:6 concrete was employed.

CHAPTER XXXI.

ARCH BRIDGE CENTERS AND METHODS OF CONSTRUCTION.

Substantial centers are necessary in the construction of concrete bridges, as it is desirable to support the structure without any possibility of the lagging sinking or sagging when the concrete is being put in place or before it has set.

In the construction of girder bridges forms similar to those used for floors are used, and need no further consideration in this place. Forms for concrete arch bridges resemble in many particulars those used in the construction of stone bridges. Not only must the arch ring be supported, but provision must be made to retain the concrete for the spandrel construction. The supports for the forms should be as unyielding as possible. If the subsoil is sufficiently firm, mud sills may be put down, and the bents or struts rested directly on them. Temporary masonry foundations are sometimes built, upon which to rest the supporting bents. Piles are also used, and are driven, if possible, to a firm bearing, so that no settlement will occur when the weight of the structure is brought upon them.

In general, centering may be divided into two classes. In the first of these, struts or braces support the timbers or ribs carrying the lagging at every joint. These struts or posts are braced together to form transverse bents, which are spaced at convenient distances apart along the central line of the bridge. At times the bents are braced together longitudinally. In the second form trusses are used either to carry the lagging directly or to support short braces carrying the ribs which support the lagging. These trusses may be carried on bents or masonry supports at the ends. There is no distinctive division between the two classes. Examples will be given of both types of centering used on important construction.

Where trussed centering is used a slight camber is usually provided, amounting to from $\frac{1}{1000}$ to $\frac{1}{800}$ of the span. The ribs upon which the lagging rests directly are usually formed by spiking together several planks, so as to break joints. The top edges of the plank are cut to a curve parallel with the intrados, and the thickness of the lagging below or inside of the same so the top of the lagging will coincide with the intrados. Two-inch plank sized to 1¾ in. is usually employed for lagging. Supporting ribs should not be spaced more than 3 to 4 ft. centers when such plank is used.

Transverse timbers spaced a foot or more apart are sometimes used on top of the ribs, and a tight longitudinal floor of tongue and groove flooring used to form the lagging. Care should be taken to prevent the concrete from sticking to the lagging. Any of the methods already described may be employed, as the use of paper, canvas, soft soap or an oil coating. Sometimes a layer of clay is used in the place of any of the above-mentioned substances.

Wedges or sand boxes are used under the braces supporting the longitudinal ribs, in order to facilitate the striking of the centers. It is customary in Europe at times when stiff reinforcing trusses are used, and in some forms of Melan construction, to carry a portion of this centering on the ribs. A hanger used for this purpose in the construction of the Laibach hinged arch bridge is shown in Fig. 610.

The spandrel, together with face wall centering, may be built in place at the same time the arch ring centering is put in place. The vertical timbers supporting the lagging are tied together at the top by cross ties. Particular care should be employed when sand boxes are used, for if any foreign substance is allowed to enter them, or they are not properly sealed, trouble may be experienced, either in their settling too soon or failing to settle at the proper time. Care must be exercised to secure a good quality of sand to be used in the boxes. Mr. Thacher states that he experienced some trouble in the use of sand boxes in the construction of the Jacaquas River Bridge in Porto Rico. It was found, upon examination, that a poor quality of sand was the cause of the trouble.

In important construction, where long span trusses are used to carry the centering, it is sometimes necessary to determine the amount of probable deflection in the truss. This may be done by using the usual formula for deflection, which may be obtained from any work on mechanics.

Figure 646 shows design of centering for a 50-ft. masonry arch by Mr. J. H. Milburn, for use in a stream where it was desirable to obstruct the waterway as little as possible. By the use of tight lagging and an end wall this center may be adapted to the construction of a concrete arch. Sizes of all materials are marked on the drawing.

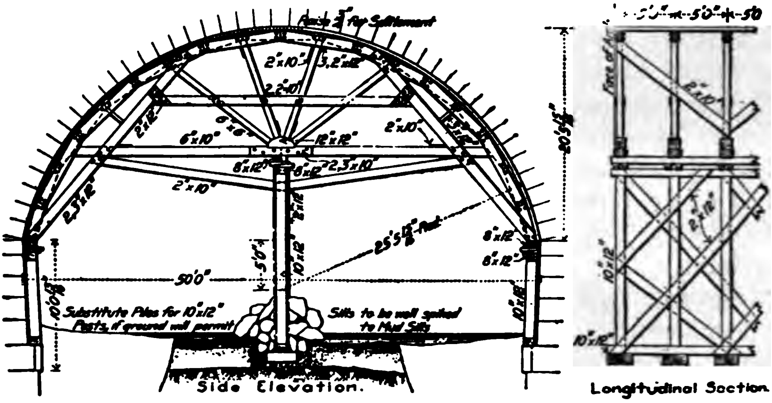


Fig. 646.—Center for 50-Ft. Arch.

A centering of extremely simple framing is shown in Fig. 647. This centering was used in the construction of a reinforced concrete arch bridge, constructed at Plainwell, Mich., in 1903. The heels were supported on benches constructed upon each pier and abutment foundation, and each center was supported at the

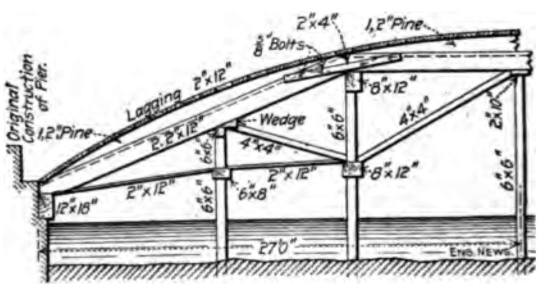


Fig. 647.—Center for 54-Ft. Arch at Plainwell, Mich.

panel points by twelve temporary piles. The size and framing of the timbers are shown in the drawing.

The centering for the Austell Bridge, described on page 781, is shown in Fig. 600. The falsework for each arch consisted of seven transverse bents of round timber, six pieces to the bent. These rested on sub-sills, the full length of the bent, and to which the posts were toenailed. On these posts rested as plates two 3 × 12-in. timbers, which formed the base on which rested the struts supporting the ribs. Transversely and resting immediately on the posts were placed two 3 × 12-in. pieces, on which were placed two wedges 2 ft. long, having bases 12 × 7½ ins., and having their small ends 3 ins. thick. The struts supported six ribs, each consisting of pieces of 2 × 12-in. Georgia pine, cut to different radii. These timbers were doubled, so that when the rib was completed, it was 4 ins. thick and 12 ins. deep. The lagging was 4 × 4-in. stuff, sized to 3⅞ ins. After this was laid in place, tongued and groove dressed flooring was laid on it, forming a close, smooth surface.

In constructing the Luten Bridge, described on page 785, the pavement of the bridge was constructed first, and the forms for the arching were erected upon this pavement. The centering consisted of 2 × 12-in. pieces sawed to the circular arc of the arch and supported on uprights, transmitting the load directly to the pavement. An unusual form of centering was used in the construction of this bridge, the idea being to dispense with the usual sand-boxes and wedges used for striking the centers. Slender uprights designed for columns by Gordon's formula were used. The sections were too light to support the load of concrete and earth filling that was to go upon them without bracing. Cross bracing connected at two points, dividing the uprights approximately into thirds, was used, and the uprights then had sufficient rigidity to support the loads coming upon them without buckling. To strike the centers it was only necessary to remove the sway bracing, and the uprights would buckle, letting the arch down easily without jar until the full stresses in the rib should be developed. About 30 days after the completion of the arch the bracing was removed from the uprights, beginning at the ends of the span and working toward the middle. As the bracing was being removed, the uprights gradually yielded, buckling from 4 to 6 ins., and allowing the arch to settle about ¼ in. from the crown. The resistance of the uprights gradually decreased as the arch assumed its loading, so that the effect on the arch was practically the same as when releasing wedges or sand-boxes are used; but in this case the operation is under much better control. After the uprights had buckled, they were removed, one at a time, selected at intervals.

The design of the uprights was in detail as follows: The uprights at mid-span were about 15 ft. long, and spaced 7 ft. on centers across the stream and 3 ft. across the roadway. Each upright then was required to support a loading of concrete 7 ft. × 3 ft. × 26 ins., besides the earth filling 1 ft. in depth over

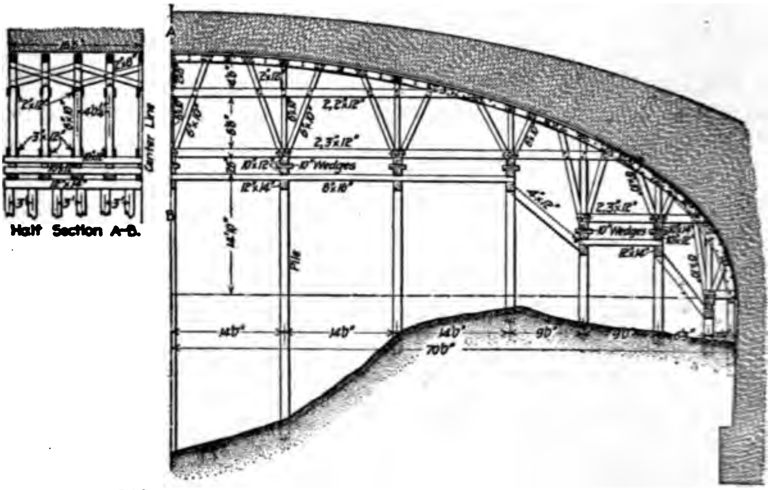


Fig. 648.—Center for Big Muddy Bridge, Illinois Central R.R.

the rib, making a total load of about 9,000 lbs. Gordon's formula for struts of free ends:

$$P = \frac{f A.}{1 + \frac{l^2}{125 h^2}}.$$

When P is the total load = 9,000 lbs.
A is cross sectional area of strut.
f is the fibre stress for oak = 1,600 lbs.
l is the unsupported length of post in inches.
h is the diameter of strut in inches.

It was found that for a length of 15 ft., a 7 × 7-in. upright would be required to satisfy the formula; but for a length of 5 ft., which would result from bracing each strut at two points, a 4 × 4-in. timber would satisfy the formula. Therefore, 4 × 4-in. posts supported at two points were used for the longest uprights.

Centering for a 140-ft. span concrete bridge over the Big Muddy River for the Illinois Central Railway is shown in Fig. 648. Details of construction are clearly shown in the illustration.

The centering for the 150-ft. concrete arch bridge, Connecticut Ave., Washington, D. C., is shown in Fig. 649. These centers were, for the most part, founded on piles, although concrete foot-

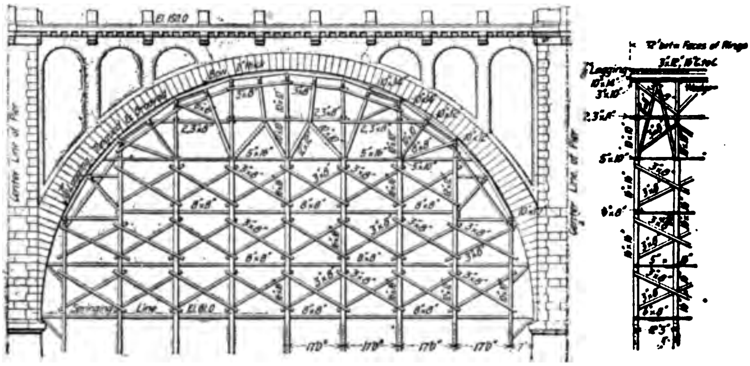


Fig. 649.—Center for Connecticut Avenue Bridge, Washington, D. C.

ings were used when the footings rested on hard rotten rock. With the exception of a few struts, which were of Virginia pine, and the sills, caps and wedges, which were of white oak, the centers were made of Georgia pine. The centers were laid

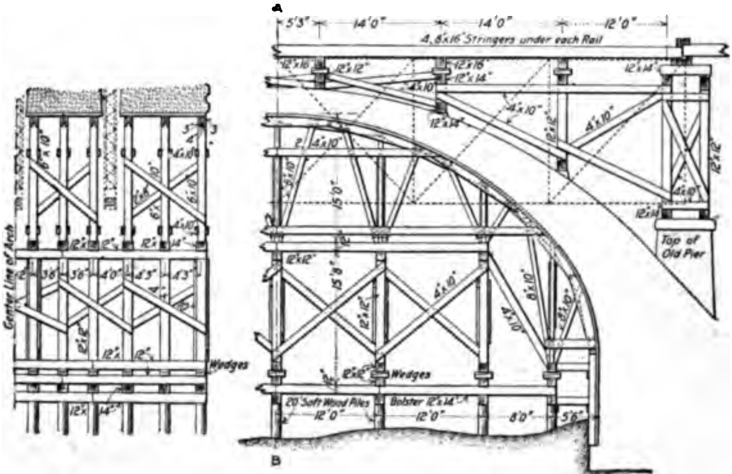


Fig. 650.—Center for Arch Bridge at Plano. Ill.

out full size on a large floor, and a pattern made for each piece. The lagging was ⅞ in., supported by 3 × 12-in. joists. The sizes of all framing are given on the drawing.

Figure 650 shows centering used for a double-track concrete bridge at Plano, Ill., for the Chicago, Burlington & Quincy Ry. This bridge was designed to carry a live load of 1,000 lbs. per sq. ft., and has sufficient strength without reinforcement, although the arch was reinforced with rows of ⅞-in. corrugated bars, 12 ins. apart, each row having four bars, 4 ins. and 6 ins. from the intrados and extrados respectively. The bars of each pair were staggered to give a spacing of 6 ins., and between them ran ¾-in. transverse bars about 2 ft. apart. The half section of this bridge is shown in Fig. 651. The thickness of arch arch ring at the crown is 3 ft. The arrangement of steel reinforcement is also shown in Fig. 651.

This bridge is of particular interest on account of its being constructed while the old steel trusses remained in place, and

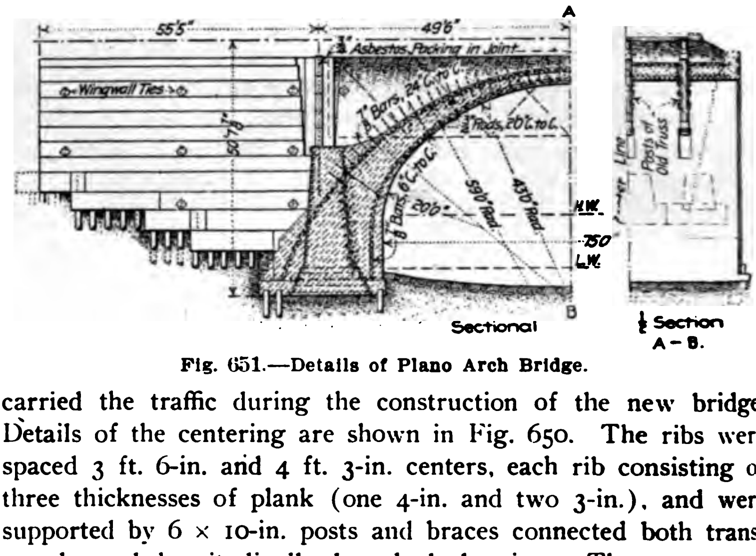


Fig. 651.—Details of Plano Arch Bridge.

carried the traffic during the construction of the new bridge. Details of the centering are shown in Fig. 650. The ribs were spaced 3 ft. 6-in. and 4 ft. 3-in. centers, each rib consisting of three thicknesses of plank (one 4-in. and two 3-in.), and were supported by 6 × 10-in. posts and braces connected both transversely and longitudinally by plank bracing. The posts and braces rest on 12 × 14-in. girts, with 12 × 12-in. caps, sills and posts. The foundations for the centering consisted of 20-ft. piles, with 12 × 14-in. caps and bolsters, the latter carrying 12 × 12-in. sills directly under the sills of the centering. Between the two sills were fitted the oak wedges, 10 ins. wide and 24 ins. long. The lagging was 5¾ × 2⅞ ins., dressed on the upper face and both edges. The forms were of planking dressed on the inner face, and the high forms for the wing walls were supported from the outside by a system of X bracing transverse to the face of the wall.

The steel members of the old trusses were encased in wooden boxing. The holes in the concrete formed by this boxing were smaller at the bottom than the top, and after the removal of the old trusses were filled with concrete.

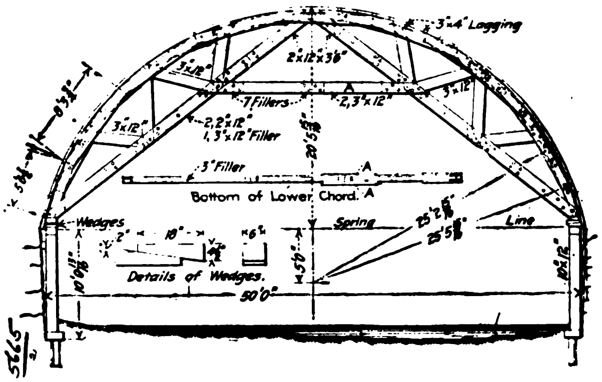


Fig. 652.—Center for 50-Ft. Arch, B. & O. R.R.

Figure 652 shows a trussed center designed for a 50-ft. arch span for the Baltimore & Ohio R. R.

The centering for the Gruenwald Bridge, shown by Fig. 653, was designed with great care, to minimize as much as

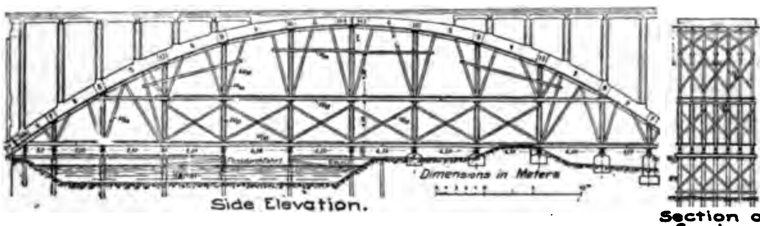


Fig. 653.—Center for Gruenwald Bridge.

possible the deflection and to prevent undue stresses in the concrete before it had become thoroughly hardened. Figure 653 shows an elevation and cross-section of the centering used. Piles were driven and masonry footings built to support the bents carrying the posts and struts supporting the timbers on which the lagging rests. The centering rests upon sand-boxes placed at each panel point. Details of the sand-boxes are shown in Fig. 654. The box is made of wrought iron, and the plunger of oak.

The hinges and the reinforced concrete hinge blocks were placed on centering before any concreting was done, both rows of hinge blocks resting on the centering, thereby preventing any possible slip of one-half of the hinge relative to the other while the bridge is under construction.

The concreting of the arch ring was carried on in separate transverse courses, whose distribution was so planned as to avoid the formation of long, continuous sections up to as late a time as possible. In Fig. 653 the order of concreting is indicated by numbers placed on the arch sections. The concrete in each full panel was placed before the joining blocks over the posts were put in, and the order of the latter is such as to make the

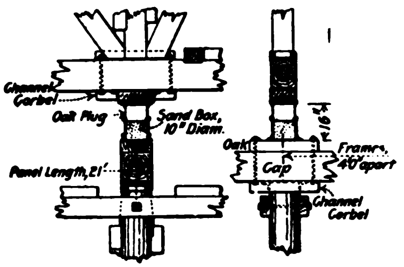


Fig. 654.—Details of Sand Box, Gruenwald Bridge Center.

three last closures at crown, quarter points and skewbacks in succession. A 1:2:4 gravel concrete was used. When the arch concrete was about three months old, the centers were slacked in the following manner: About 15 cu. ins. of sand was drawn out of each sand-box of the middle row of each span. After plugging the openings, the boxes were tapped with a hammer, causing the plungers to settle down. Then the adjoining rows on either side of the center were treated in the same manner, and so on until all boxes had been slackened. Returning now to the middle, the same procedure was repeated again and again, until the levels on the bridge showed the elastic compression was going out of the centers, i. e., that the load was nearly all on the arches themselves. The centering was then removed. It is stated that the centering was built with a camber of 4 ins., and that under full load the deflection was about $^{11}/_{16}$-in.

The drop of the crown of the arch was about ¼ in. and ⅜ in. respectively for the two spans.

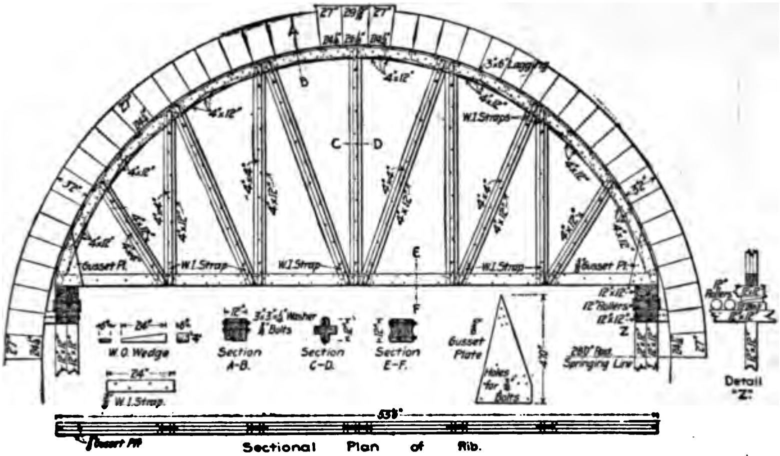


Fig. 655.—Center for Raritan River Bridge, Pennsylvania R.R.

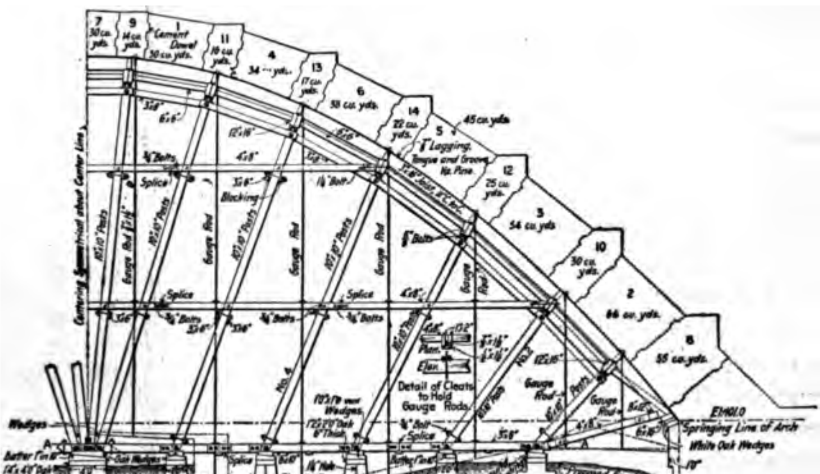


Fig. 656.—Side Elevation of Center for Piney Creek Bridge.

The truss arch center used in the construction of the Raritan River stone arch bridge of the Pennsylvania R. R., at New Brunswick, N. J., is a good illustration of the use of trussed arch

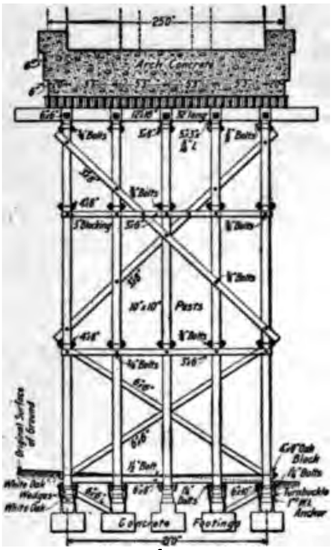


Fig. 657.—Transverse Section of Center for Piney Creek Bridge.

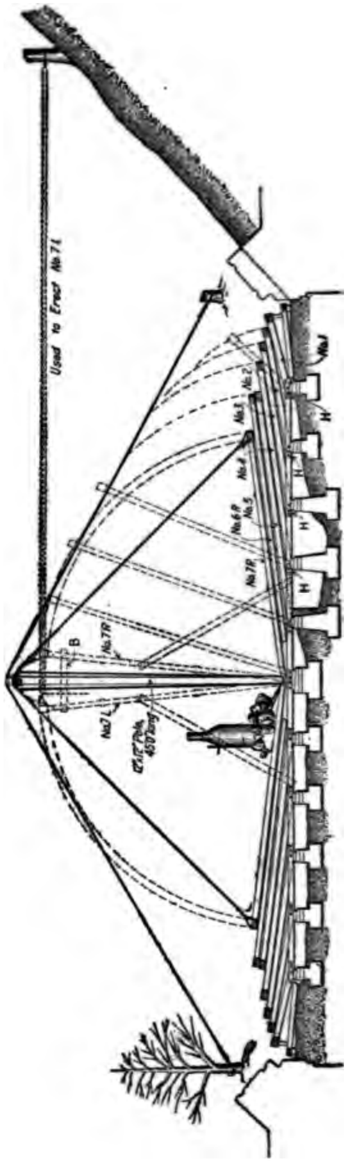


Fig. 658.—Method of Erecting Centers for Piney Creek Bridge.

centering. Figure 655 shows details of trussed ribs, sizes of members, etc. Yellow pine was used throughout. Wrought iron straps and steel gusset plates were used for the connections. The trusses were supported on bents. Wedges were used to support the trusses at proper elevation. Trusses of similar construction may be used for concrete arch construction.

The arch centering used for the Piney Creek Bridge, Washington, D. C., is of unusual design. A half side elevation of this centering is shown in Fig. 656, while a transverse section is shown in Fig. 657. The method of erection of the centering will be understood from Fig. 658. Special details of this centering are shown in Fig. 659. Referring to Fig. 656, it will be seen that the center consists of 14 radial bents, carrying joists spaced 11 ins. apart on centers, and covered with tongued and grooved lagging arranged in panels, as shown in Fig. 659. The bents rest on and are anchored to concrete footings or pedestals, one under each post. The posts, caps and braces were framed into the bents on the ground in the position shown in Fig. 658. When all was ready to hoist into place, the first bent to the left of the center (bent 7 L) was pulled into place by means of a tackle running from its top or cap to a tree or deadman on the high ground. As soon as bent 7 L was in place, tackle was attached

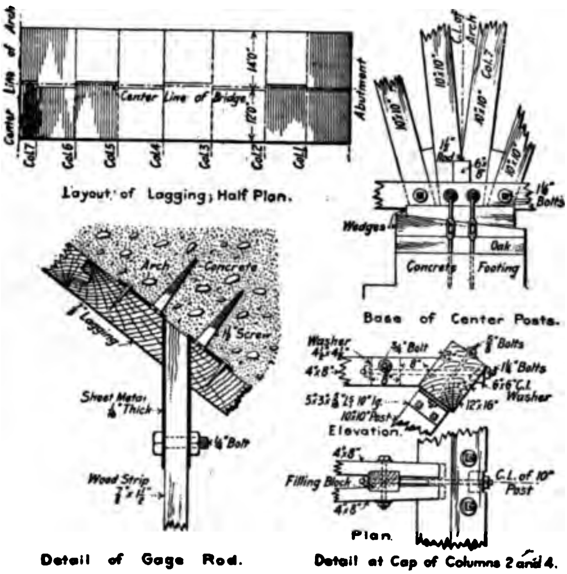


Fig. 659.—Special Details of Centers for Piney Creek Bridge.

to its cap, and bent 7 R pulled into position and the two braced together. All the other bents were then successively hoisted into place and fastened together. A reverse operation was employed when it was desired to strike the centers.

A reinforced concrete bridge recently constructed near Belvidere, Ill., is of unusual interest, as it was constructed without centering. The methods used in the construction of this bridge may be applied in situations where centering will be difficult to construct and when built may be endangered by floods. This bridge is a single track electric railway bridge, 350 ft. long, consisting of four arches of 81 ft. clear span. Each arch has a circular intrados with a radius of 83.36 ft. and a rise of 10.5 ft. The arches each have two longitudinal arch ribs 8 ft. 10 ins. apart on centers. The ribs are 2 ft. 6 ins. wide their entire length, but vary in depth from 3 ft. at the crown to 4 ft. 6½ ins. at the haunches. Each rib carries a 12-in. spandrel wall centrally

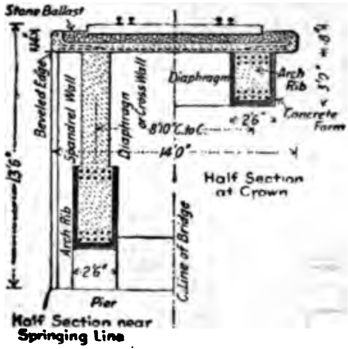


Fig. 660.—Half Transverse Section of Bridge at Belvidere, Ill.

placed above the rib and built up to the horizontal plane through the extrados of the arch at the crown. Transverse beams span between and connect the two arch ribs. On these beams are built 12-in. cross walls flush at the top with the spandrel walls, and on the latter and the cross walls are carried the floor slab of the

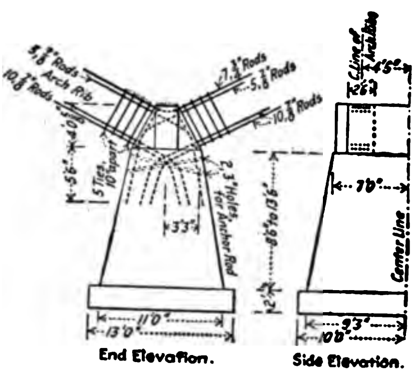
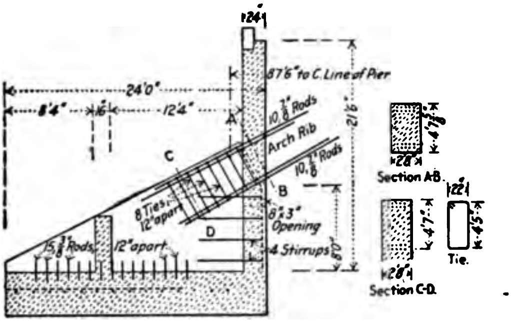


Fig. 661.—Details of Piers for Belvidere Bridge.

bridge. This slab is 6 ins. thick, 14 ft. wide, and has an 8-in. curb along each side to confine the ballast of the roadbed. Fig. 660 shows half cross sections at crown and near springing line.

The reinforcement of the arch ring consists of ten ⅞-in. plain round rods in both the extrados and the intrados, with ⅜-in.

stirrups placed from 12 ins. centers at the haunches to 36 ins. centers at crown. The spandrel wall reinforcement consists of ⅜-in. diam. rods spaced 12 ins., both horizontally and vertically. The floor reinforcement consists of ½-in. rods spaced 5½ ins. centers, every other rod being bent upward so as to be in the



Longitudinal Section of Abutment.

Fig. 662.—Abutments for Belvidere Bridge.

top of the slab near its outer edge where it runs up vertically into the curb. Twelve ⅜-in. round rods are also run longitudinally through the floor, and are wired to the transverse rods.

The arrangement of the reinforcement at the piers is shown in Fig. 661, while that at the abutment is shown in Fig. 662, which is a longitudinal section of the abutment.

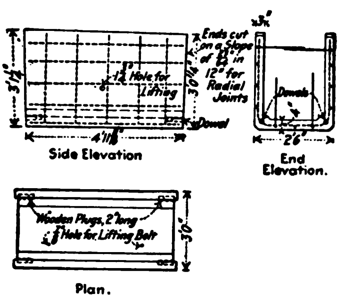


Fig. 663.—Details of Voussoir Blocks for Ribs of Belvidere Bridge.

The ribs of the arch rings were built in trough-shaped reinforced concrete forms. Each trough consisted of a series of U-shaped blocks or voussoirs, built of reinforced concrete, and made with radial joints, the face of the joints being inclined $\frac{11}{16}$-in. in 12 ins. Figure 663 shows details of the blocks, which were about 5 ft. long, 36 to 54½ ins. deep, and 30 ins. wide over all; they were 3 ins. thick in the sides, and 4 ins. at the bottom, so as to have interior dimensions 24 x 32 ins. at the crown and 24 x 50½ ins. at the haunches. The blocks weighed from 1,500 to 2,200 lbs. each. They were composed of a 1:3 mixture of Portland cement and fine crushed gravel. The reinforcement for the blocks consisted of ⅜-in. plain round rods, the horizontal rods being spaced 8 ins. centers and the vertical rods 12 ins. centers. Sockets of 1-in. gas pipe in the bottom corners served to hold steel dowels to adjust the adjacent blocks during erection and to hold them in place. Holes in the sides, at about the center of gravity, provided for handling by a crane, and holes were provided in the bottom for the lifting rods. There were 17 of these blocks in each rib, and when these were erected they formed a self-sustaining hollow arch rib of trough section, to receive the reinforcing bars and concrete of the rib proper.

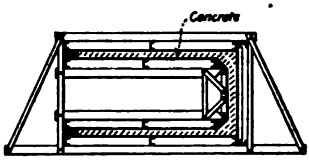


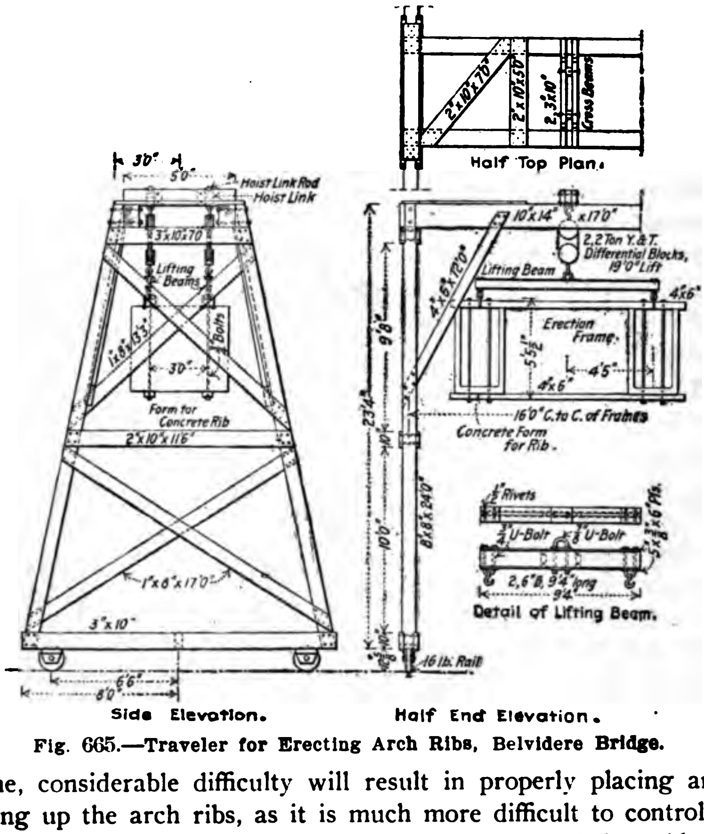
Fig. 664.—Mould for Voussoir Block, Belvidere Bridge.

These blocks or forms were cast in steel moulds built up of No. 16 sheet steel, stiffened by 3-in. channels, and held in alignment by steel angles, so connected that the moulds could be readily knocked down and reassembled. A section of one of the moulds is shown in Fig. 664.

The variation in depth of the U-shaped blocks was effected by inserting wooden stop-off strips between the inner and outer side pieces of the moulds. The arch form blocks were cast on their sides, so that one voussoir face was against the working platform on which the mould stood, while the other was flush with the top of the mould. Several of the forms had two rectangular openings in the inner wall, to provide a means of bonding the diaphragms with the ribs. The concrete for the forms hardened rapidly, it being possible to remove the steel forms after about 24 hours.

A light temporary trestle was built by driving two longitudinal rows of piles on both sides of the bridge. These rows of piles were capped with 2 x 10 transverse plank braced in both directions by 1-in. boards. On top of the piles were laid 6 × 10-in. stringers, upon which rails were spiked for the traveler. Details of the traveler are shown in Fig. 665.

The traveler spanned the site of the arches, and was fitted with two triplex blocks, each carrying a balanced beam made up of two 8-in. channels. It is stated that the inventor of this system of construction proposes to replace the traveler with a cableway in future operations. It would appear, however, if this is



Side Elevation. Half End Elevation.

Fig. 665.—Traveler for Erecting Arch Ribs, Belvidere Bridge.

done, considerable difficulty will result in properly placing and lining up the arch ribs, as it is much more difficult to control a cableway than a tramway, owing to the swaying of the cable.

The method of erection consisted first of lifting the sectional forms to a platform or a temporary bridge by a guyed derrick. The corresponding sectional forms for the two ribs of each arch were assembled in pairs, and spaced the same distance apart transversely that they were placed in the bridge. The two form sections of the two ribs were then connected by two sets of light transverse timber framing, one near each end, so as to be raised and set in place simultaneously. The cross timbers were 4 × 6-in., connected to both forms by long bolts. Transverse diagonal bracing was also fitted between the two blocks, and for sections where the diaphragms occurred the transverse forms for these were also set in place, resting upon the bottom cross timbers, so that the arch and diaphragm forms were placed in one unit.

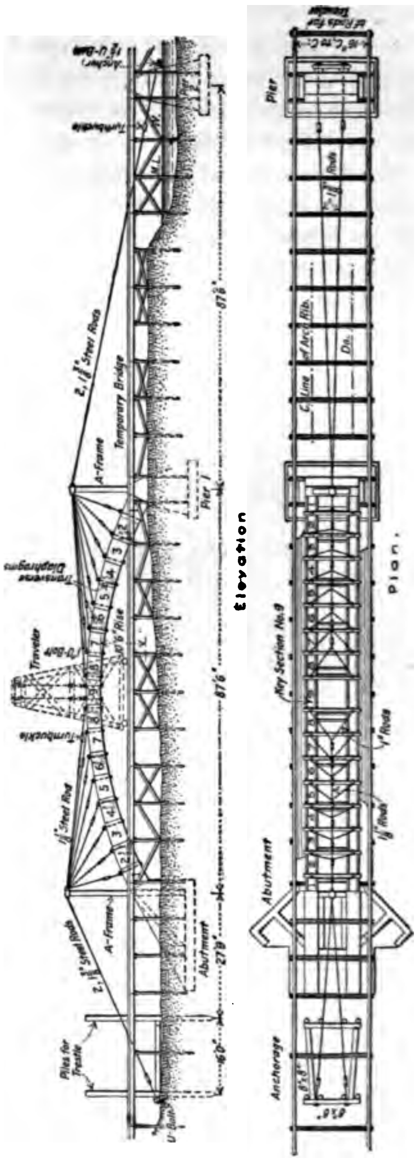


Fig. 666.—Arrangement Adopted in Erecting Belvidere Bridge.

The method of hoisting the form blocks into position is clearly shown in Fig. 666. The erection of the arch ribs was started simultaneously from each haunch, the first pair of sectional forms being lifted into place by means of the traveler hoist, and was supported at one end on the skewback and at the other end attached to steel rods suspended from an A-frame at the end of the arch. One of these A-frames was set at the abutment, and anchored back to the piers of the trestle approach to the bridge; the other A-frame for the first arch was set on the pier at the other end of that arch and anchored back to the second pier. The anchors consisted of 1⅜-in. rods in 16-ft. lengths, connected at the joints by sleeve nuts. Turnbuckles were also provided on these rods for adjusting the length of the anchors. A series of 1⅛-in. round rods, each provided with a turnbuckle, radiated from the top of each A-frame. These rods each supported two shorter 1-in. rods diverging to the sectional rib forms. The shorter rods were attached to U-bolts on the lifting frame carrying the assembled pairs of sectional rib forms, and supporting the latter until the rib forms for one arch were set. After the first forms had been placed, the second sectional form was then brought into position and supported at one end against the first form, the other end being supported by the second set of A-frame rods. The connection between the sectional forms was made by means of steel dowels placed in gas pipes, located in the bottom corners of the forms, and in some cases by wooden blocks in the inside of the forms. In this manner the sectional forms were placed in position and supported. The keystone required no support other than that supplied by the adjacent sectional forms. As soon as the keystone block had been placed the A-frame rods were slackened, and the two halves of the arch allowed to close against it. The entire series of sectional forms forming the arch then became self-supporting, and the A-frames, with their anchors and rods, were removed and used for the erection of the next arch.

To insure the proper fitting of the keystone, the faces of the skewbacks on the piers were dressed before erection was started, so as to make the angle between the face of the skewback and the horizontal plane through the springing line slightly less than that calculated. In addition the lower edge of the first sectional form was rounded, so as to keep the pressure from coming too close to the edge, and a strip of sheet lead ⅛ in. thick was placed in the joint. By means of turnbuckles on the anchor rods it was possible to raise or lower half of the arch as a unit, thereby making it possible to so adjust the two halves that the keystone could be readily set. After the removal of the A-frames and supporting rods all joints were found to be tightly closed, and the alignment was perfect.

After the sectional forms of all four ribs were placed, the reinforcement was put into position and the concrete deposited. After the arch ring had been completed the spandrel wall and floor slab were built in ordinary wooden forms.

A 1:3:5 concrete was used in the piers and abutments, and a 1:2:4 mixture for the arch rings, spandrel walls and floor. The unit stresses for the concrete were taken as 500 lbs. and 50 lbs. per sq. in., without considering temperature stresses, and 650 and 75 lbs. respectively for a variation of 40° F. The unit stresses in the steel are 10,000 lbs. per sq. in. without, or 13,000 with temperature variations. The live load for which this bridge was designed consisted of a train of 40-ton double-track electric cars. A ratio of 1 to 20 was taken between the moduli of elasticity of the steel and concrete. This system of construction is the invention of Mr. J. B. Strauss.

Placing the Reinforcement.—The methods of placing reinforcement will depend upon the kind used. When reinforcement of the Monier or loose bar type is employed, the methods used are entirely analogous to those used for floor construction. Generally the lower bars are put in wired together and blocked up, the concreting brought up, and the upper set of bars put in proper position. When ties or stirrups connect the two sets of bars, light bracing is used to hold the two sets in proper position, and knocked out as the work is brought up. When systems of the Melan and analogous types are used, great care is taken to line and fix the trusses or beams in proper position, where they are braced until the concrete is placed.

Concreting.—Concreting for arch bridges may be placed in one of two ways. The arch ring may be divided into convenient longitudinal sections, which can be concreted in a single day, and each section concreted separately. Again, the arch ring may be divided into a number of transverse sections, and each section concreted separately. These transverse sections are bounded or limited by radial planes, so that all pressure brought upon the planes of juncture will be normal to them. It is generally believed that the latter method is preferable, as all lines of weakness are normal to lines of pressure. Again, any settlement of the centering will not weaken or fracture the concrete before it has set, especially if the last sections concreted are at or near the springing lines. When the arch is concreted in longitudinal sections there is danger of the sections separating along the planes between adjacent sections. Again, undue strains may be brought upon the concrete from a variety of causes before it has set sufficiently to attain its full strength.

Striking Centers.—As a rule, centers should not be struck until the concrete has set for at least 30 days, and it is desirable that a longer period should elapse if possible; from two to three months if possible. In a number of cases centers have been struck at the end of two weeks, but this is a dangerous practice, and should not be taken as a safe precedent. The operation of striking the centers should be carried out carefully, the centers being lowered as slowly as possible, and the bridge allowed to take its stresses gradually.

CHAPTER XXXII.

BRIDGE FLOORS.

When used for bridge floors, the reinforced concrete slab usually rests either directly upon the top flange of the girders

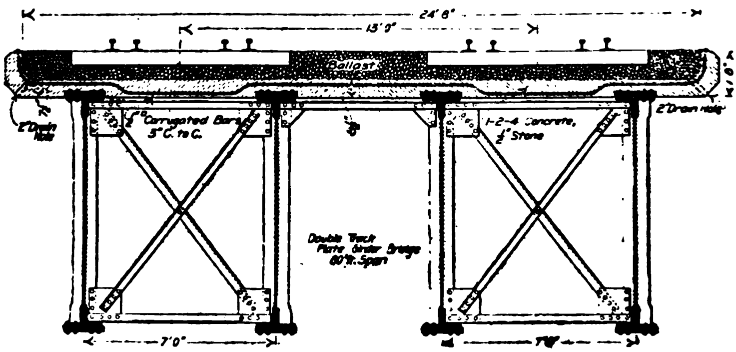


Fig. 667.—Floor for Deck Bridges, C., B. & Q. R.R.

when used for a deck bridge or upon floor beams and girders when used on a through bridge. Figure 667 shows the cross-section of a concrete floor slab of trough section used by the Chicago, Burlington & Quincy Ry. for deck bridges. As will be seen, the trough section retains the standard ballast, the cross-ties being placed in the usual manner. The floor slab is 8½ ins. thick. The transverse reinforcement consists of ½-in. square corrugated bars spaced 3 ins. centers. No longitudinal reinforcement whatever is used. This type of floor slab is used for both single and double-track bridges. When used for double-track floors, one track is put in at a time, the two being connected by the transverse rods.

Figure 668 shows the reinforced concrete floor used for the Chicago & Eastern Illinois Ry. over the Embarras River at Villa

Grove, Ill. The slab is 10 ins. thick at the center over each track, and tapers to 7 ins. at each side, and is reinforced with ¾-in. square transverse corrugated bars spaced 4½-ins. centers at the bottom of the slab, and spaced 6-ins. centers at the

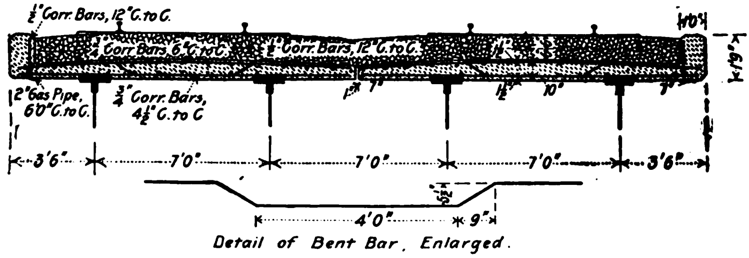


Fig. 668.—Floor for Deck Bridge, C. & E. I. Ry.

top of the slab, while the top and bottom longitudinal rods are ½ in. square and spaced 12-ins. centers.

The type of floor used by the Wabash Ry. is shown in Fig. 669. The reinforced slab is supported by 9-in. 25-lb. beams spaced

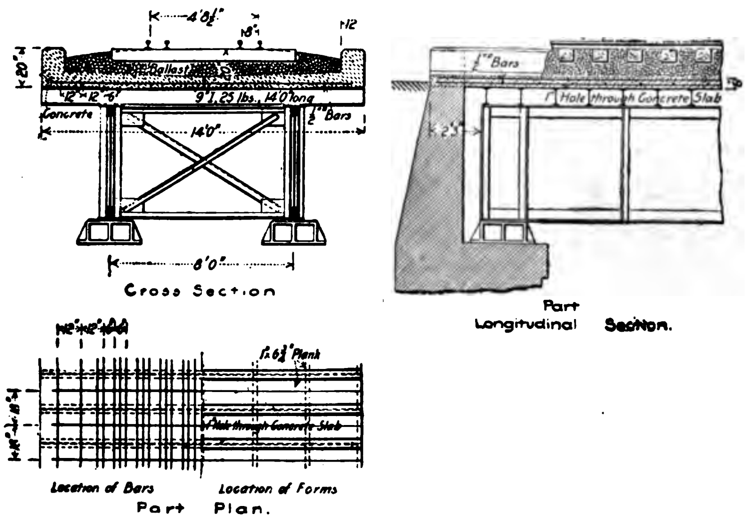


Fig. 669.—Floor for Deck Bridges, Wabash Ry.

18-ins. centers. The trough section retains the ballast in the usual manner. The slab is 6 ins. thick at the center, and increases to 8 ins. at the sides, and is reinforced with ½-in. square corrugated bars. In the longitudinal direction they are placed 12 ins. apart and 1¼ ins. below the top of the slab, and in the bottom they are spaced 6-ins. centers and placed $1\frac{1}{2}$ ins. above the bottom of the slab. In the transverse direction the rods are spaced 18-ins. centers, and rest on the bottom longitudinal rods.

For through bridges a type of floor similar to that just described is used. The reinforced slab rests upon transverse floor beams spaced 18-ins. centers, and forms a trough floor to support the ballast and cross-ties. The sides of the slab extend up to form curb walls against the girders, and enclose the gusset plates. The details of this construction are clearly shown in Fig. 670.

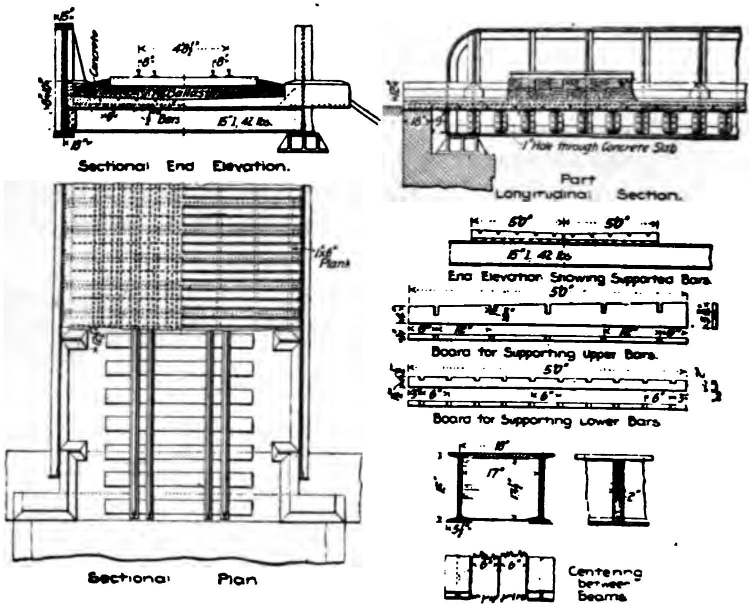


Fig. 670.—Floor for Through Bridges, Wabash Ry.

The forms necessary for the construction of this type of floor are quite simple, consisting of 6-in. plank flush with the tops of the beams, and supported by cross pieces between the beams. When the concrete has set these cross-pieces are knocked out, allowing the lagging to drop out. The forms which hold the reinforcing rods in place during the placing of the concrete are shown in the drawing.

For situations where there is a limited clearance beneath the bridge floor the type of floor shown in Figs. 671 and 672 is used. In this case the I-beams are punched in the webs near the bottom flange. Rods spaced 6-ins. centers are threaded through these holes. Transverse rods, one on each side of the I-beam, and one between them, are laid on the longitudinal rods and the concrete slab of the same section, as given for the previous designs, is laid on the bottom flange of the I-beam, and built around them, covering them, and leaving a rectangular trough about 12 ins. square between them. In this trough the ballast and ties are placed. Holes are made in the center of the slab between the

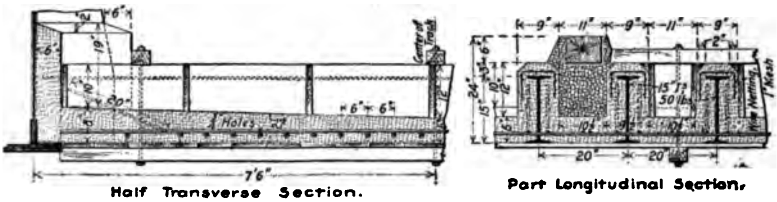


Fig. 671.—Shallow Trough Floor, Wabash Ry.

I-beams, to drain off water which is collected in a gutter placed in line with the holes, and carried by down spouts to the street curbs.

Figure 673 shows a bridge built of old bridge stringers embedded in concrete beams, and carrying a floor of concrete all reinforced with longitudinal and transverse rods. The floor extends over the parapet or end wall of the bridge abutment, and the top of this wall was coated with soap to prevent adhesion of the concrete when laid, and the floor is therefore free to

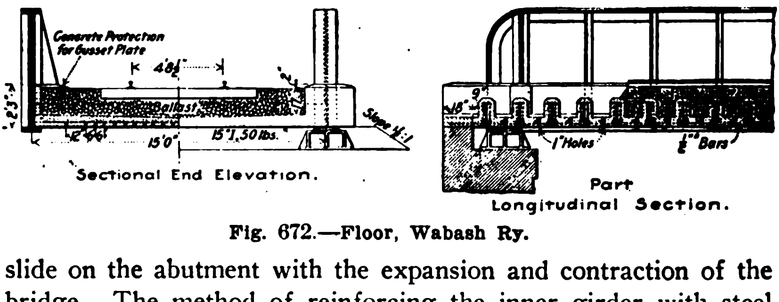


Fig. 672.—Floor, Wabash Ry.

slide on the abutment with the expansion and contraction of the bridge. The method of reinforcing the inner girder with steel bars embedded in concrete should be noted, as this method has been successfully employed in a number of cases for strengthening old bridges.

Figure 674 shows part transverse section, and Fig. 675 longitudinal sections of reinforced concrete floor construction of highway bridges over the track depression of the Chicago & Northwestern Ry., at Milwaukee, Wis. Expanded metal was used for the reinforcement.

Reinforced concrete is being used in the construction of the roadway of the Philadelphia Rapid Transit Elevated Ry. The floor consists of the usual Z-bar and plate trough floor. The de-

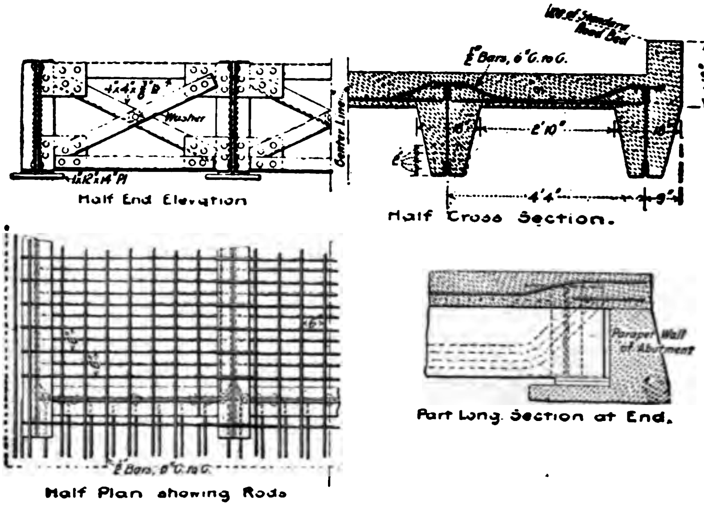


Fig. 673.—Trough Floor Supported by Old Girders Strengthened with Reinforced Concrete.

pressions are filled with concrete, which is filled in until it extends 4 ins. above the top of the trough. This floor is reinforced against cracks caused by shrinkage or temperature strains by ⅜-in. corrugated bars spaced 18-ins. centers. The ballast is held by the concrete, which forms a trough. Figure 676 gives cross-section of this floor.

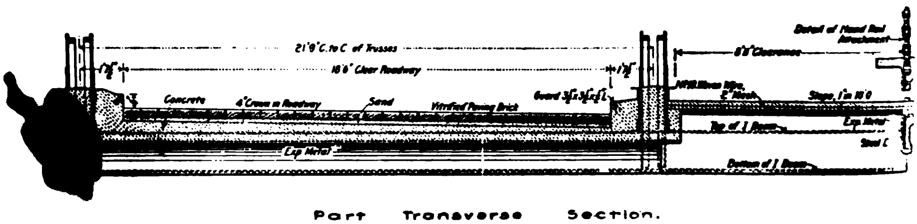


Fig. 674.—Highway Bridge Floor, Chicago & Northwestern Ry.

Figure 677 shows details of a reinforced concrete floor used on a plate girder highway bridge at South Bend, Ind. The roadway is carried by four longitudinal girders, spaced 12-ft. centers. The floor beams are 15-in. 42-lb. I-beams, spaced 6-ft.

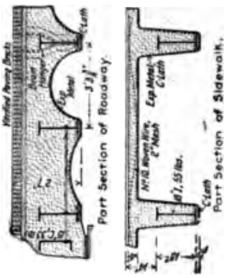


Fig. 675.—Highway Bridge Floor, C. & N. W. Ry.

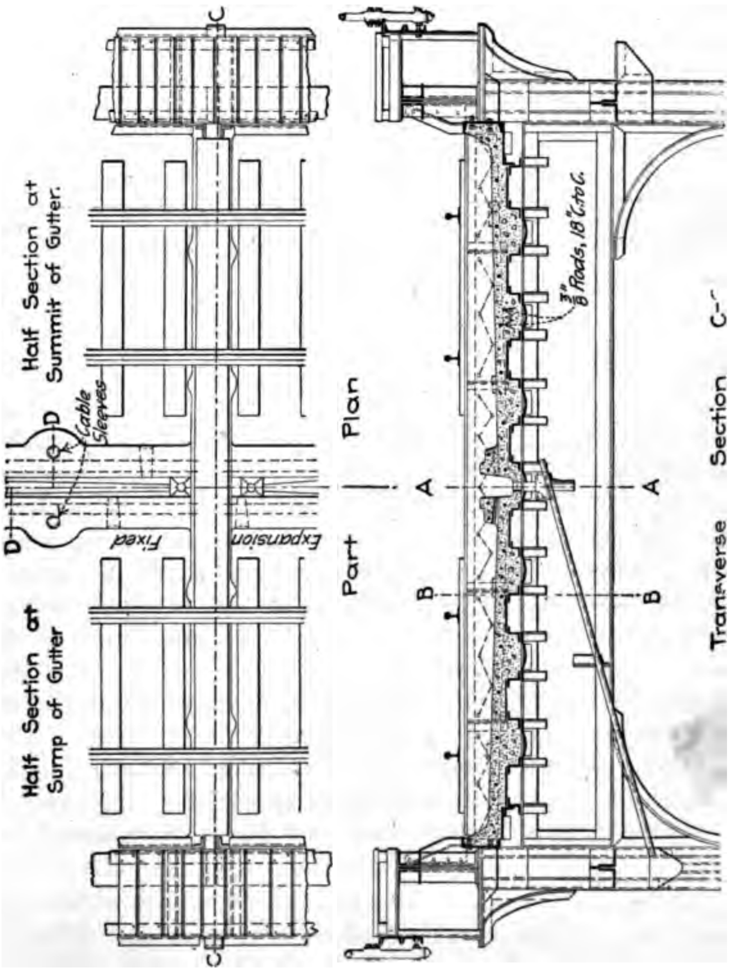


Fig. 676.—Philadelphia Rapid Transit Elevated Ry. Floor.

centers. The sidewalks are on brackets cantilevered out from the roadway girder on each side, and are spaced 12-ft. centers. The concrete sidewalk slab spans between the brackets, and has a thickness of 5 ins. of 1:3:6 gravel concrete and 1-in. surface coat of 1 to 1 mortar. This slab is reinforced with ½-in. Thacher bars 4-ins. centers placed longitudinally. The roadway

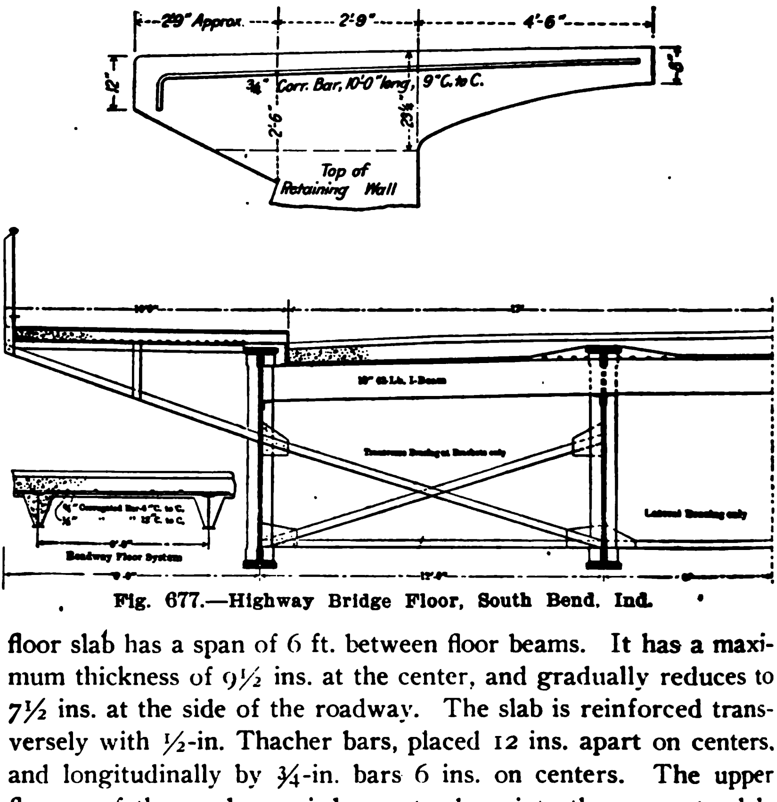


Fig. 677.—Highway Bridge Floor, South Bend, Ind.

floor slab has a span of 6 ft. between floor beams. It has a maximum thickness of 9½ ins. at the center, and gradually reduces to 7½ ins. at the side of the roadway. The slab is reinforced transversely with ½-in. Thacher bars, placed 12 ins. apart on centers, and longitudinally by ¾-in. bars 6 ins. on centers. The upper flanges of the roadway girders extend up into the concrete slab, and the transverse rods are bent up over their flanges.

Figure 677 also shows a cantilever sidewalk on a retaining wall used at one end of the South Bend bridge; ¾-in. corrugated bars spaced 9-ins. centers were used for reinforcement.

CHAPTER XXXIII.

BRIDGE PIERS AND ABUTMENTS.

Piers.—Concrete is used in many cases to replace stone for pier construction. When no reinforcement is used, the sizes and general proportions follow the usual practice employed for stone piers, the masonry yardage used for the two not differing materially. When reinforced concrete is employed, a considerable saving of concrete, with a corresponding reduction in cost, will result. The reduction in masonry may be obtained either by reducing the size of the pier or using the same size pier as when stone is used, and making it hollow, with reinforced walls. Interior cross walls or diaphragms may be used if necessary. The interior open space may be filled with broken stone, sand or gravel, or may be left open if the pier is designed so that it will possess sufficient stability. In the latter case considerable reduction in the load on the foundation will result.

The top slab forming the coping should have sufficient strength to support the loads brought upon it, and transmit them to the side and interior walls, which in turn transmit the loads to the foundation. It is well to make the coping slab somewhat heavier than theoretical requirements dictate, as the shock of passing trains bring dangerous and indeterminate stresses upon it. Care should be taken to provide sufficient area to the wall so that the safe bearing value of the concrete will not be exceeded. The interior faces of the walls may be vertical, but outside faces should have a batter of at least ½-in. to the foot. Care should be taken to provide sufficient reinforcement for the side walls to enable them to resist the pressure of the stone or sand filling, as well as any external loads or blows which may come upon them. The cutwater may be strengthened by a rib or wall extending back into the pier.

When it is necessary to spread the foundation either to reduce the pressure on the soil or increase the stability of the pier, a reinforced slab with strengthening ribs should be used in a manner entirely analogous to that employed for retaining wall footings.

Pier, K. C., M. & O. Ry.—Figure 678 shows design of a hollow reinforced concrete pier 20 ft. in height used on the Kansas City, Memphis & Omaha Ry.

Illinois Central R. R. Piers.—Figure 679 shows plan, elevations and sections of a hollow reinforced concrete pier used for a 450-ft. draw span at Gilbertsville, Ky., by the I. C. R. R. The general dimensions of the pier are shown on the drawing. The interior hollow space is 30 ft. in diameter, and is domed at the top, while the load is uniformly distributed to the base by the

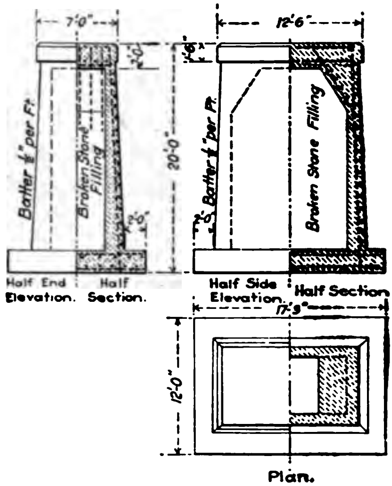


Fig. 678.—Hollow Pier, K. C., M. & O. Ry.

inverted dome at the bottom. The side walls are 8 ft. thick, and are composed of a 1 : 2½ : 6 concrete. The draw span carried by this pier is rim bearing. The entire dead and live loads from the trusses are carried by a drum, which rests on coned wheels bearing on a cast-iron track 38 ft. in diameter, centered over the center line of the circular pier walls. The pier rests upon piles. The reinforcement for the pier consists of ¾-in. corrugated bars spaced 2-ft. centers and placed on the sides, top and footings, as shown in Fig. 679.

The fixed spans of the Gilbertsville Bridge are double-track bridges of 300-ft. span. Figure 680 shows side and end elevations, vertical section and plan of pier 68 ft. in height, carrying the 300-ft. fixed span trusses. The coping detail is also shown in Fig. 680. The nose of the pier is of segmental form, with an angle of about 90°. It is stated that this form of nose will turn drift, and will turn the water so as to prevent cross currents and eddies below the pier. The above piers were designed to sustain, besides their own weight and that of the superstructure of a double-track bridge and its train loads, a tractive force equal

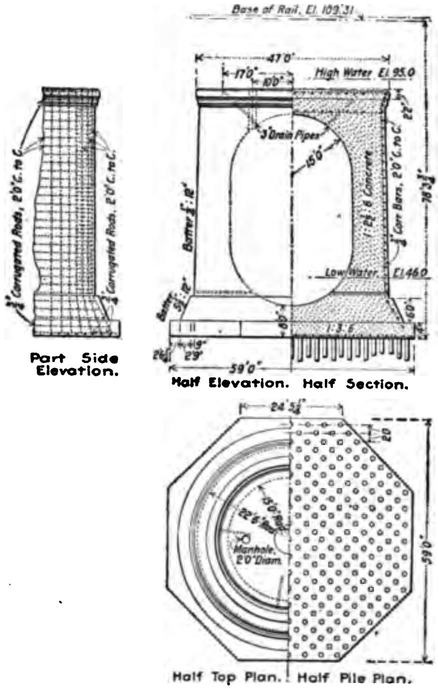


Fig. 679.—Hollow Pivot Pier, Gilbertsville Bridge, Illinois Central R.R.

to 0.2 of the double-track train loads applied longitudinally to the rail, and a wind load against the exposed surface of the bridge and train equal to 150 lbs. per lin. ft. applied in the plane of each chord, and 300 lbs. per lin. ft. (against the side of the moving train) applied 7 ft. above the rail. The above loads produce a maximum compression at the bottom of the shaft of the pier of 18.2 tons per sq. ft. (252 lbs. per sq. in.), and a maximum tension of 3.34 tons per sq. ft. (46 lbs. per sq. in.).

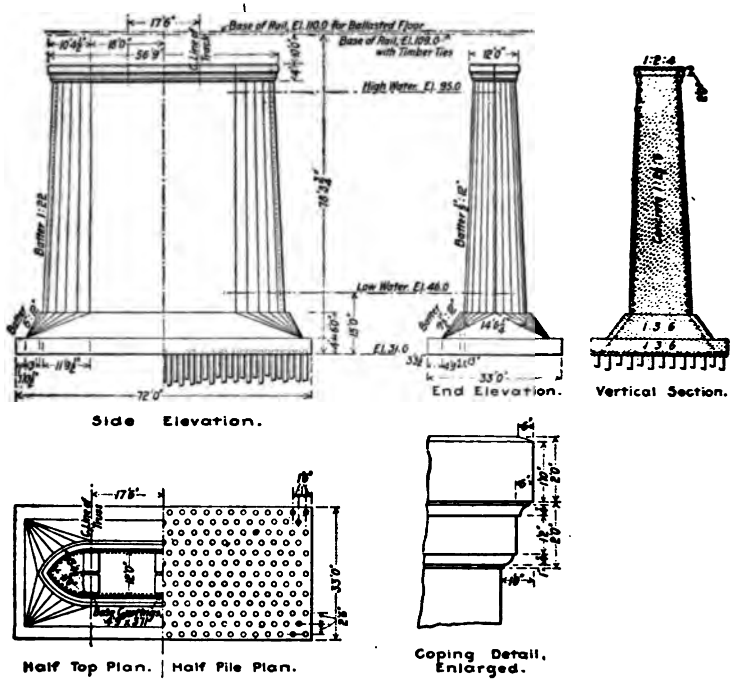


Fig. 680.—Fixed Span Pier, Gilbertsville Bridge.

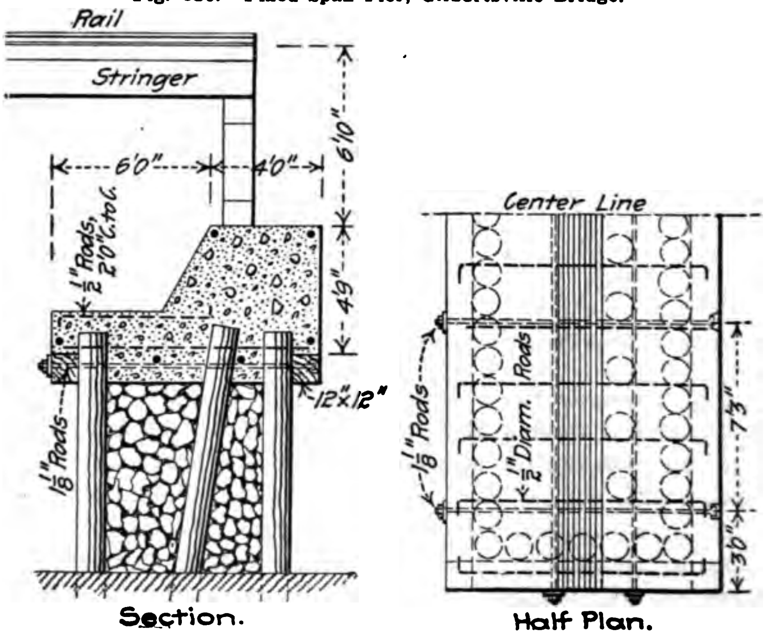


Fig. 681.—Pier for Small Drawbridge.

Reinforcing rods placed 6 ins. inside of the face of the masonry and spaced 2-ft. centers in both directions, as shown on the drawings, were used to care for any dangerous tensile stresses and insure monolithic action.

Small Draw-Span Pier and Abutment.—Figure 681 shows half-plan side elevation, half end view and section of pier for small draw span, while Fig. 682 shows half plan and section of abut-

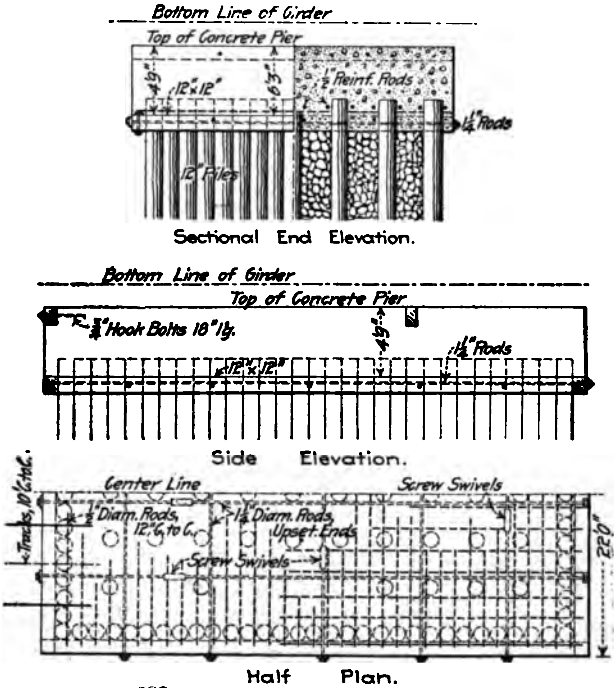


Fig. 682.—Abutment for Small Drawbridge.

ment. The size and arrangement of the reinforcing rods are shown on the drawings.

The forms consisted of 3 × 4-in. scantlings nailed to wale pieces at the base, and inch boards nailed to these. The tops of the 3 × 4's were fastened by wire to some of the inside piles, which held the forms rigidly in place. Two bridges were built as shown. The cost of construction varied slightly. The yardage for bridge No. I. was 151.6 cu. yds. for pier and 33.9 cu. yds. for the abutment, or a total of 185.5 cu. yds. For bridge No. II. the yardage was 156.7 cu. yds. for pier and 43.5 cu. yds. for abutment, or a total of 200.2 cu. yds. of concrete. The cost per cubic yard for the two jobs was as follows:

	Bridge I. Per cu. yd.	Bridge II. Per cu. yd.
Piers:		
Tugs and scows	$0.10	$0.50
Materials	3.98	4.00
Labor	1.43	1.21
Forming	.60	.55
Machine	.19	.14
Total	$6.30	$6.40
Abutments:		
Tug and scows	....	$0.52
Materials	$3.95	4.00
Labor	1.50	1.64
Forming	1.20	.83
Machine	.65	.22
Total	$7.30	$7.21

The cost for forms per cubic yard of concrete was as follows:

	Bridge I.		Bridge II.	
Erecting—	Pier.	Abutment.	Pier.	Abutment.
Per cubic yard:				
Materials	$0.18	$0.40	$0.17	$0.31
Labor	.36	.59	.32	.31
	$0.54	$0.99	$0.49	$0.62
Tearing down:				
Per cubic yard	$0.06	$0.21	$0.06	$0.21

Figure 683 shows a concrete pier for the Western Maryland R. R. Details and dimensions used are clearly shown on the drawing. The forms for the piers were made in sections 5 ft. high. When the first section above the footing course had been nearly filled, a second section was placed on top of it, and filled with concrete. A third section was successively placed on it as the concrete neared its upper edge. The lower section was then removed and lifted over the second and third sections, and placed on top of them by means of a cableway. The batter on the face of the pier was sufficient to hold the forms in place without any other support. By this means a minimum amount of lumber was used for forms.

Abutments.—When plain concrete is used for the construction of abutments, the details used are essentially similar to those used for stone abutments. If reinforced concrete is used, a great saving in materials results, with a reduction in cost of sometimes as much as 40 or 50 per cent.

For a reinforced concrete abutment the construction usually consists of a rectangular slab for a base, generally not less in width than one-half the height, depending upon the area of base necessary to distribute the load. This width of base, which is somewhat greater than that usually adopted in stone or concrete abutments, adds greatly to the stability of the structure. The spread of base is secured by extending the base plate the desired distance beyond the vertical wall on all sides. Buttresses

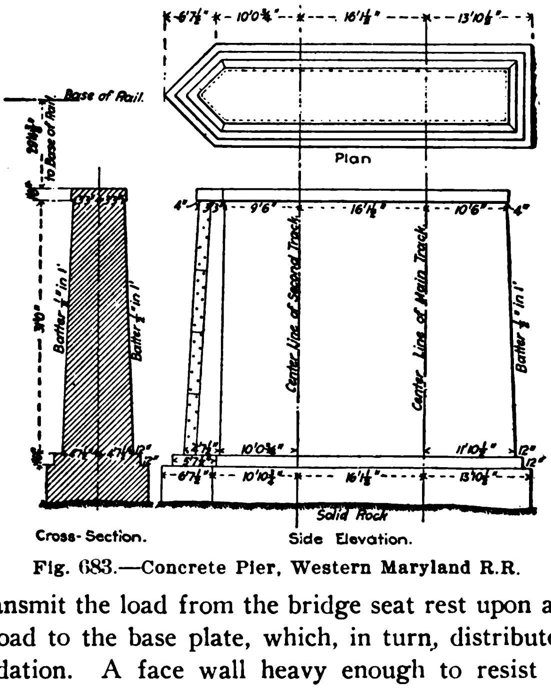


Fig. 683.—Concrete Pier, Western Maryland R.R.

which transmit the load from the bridge seat rest upon and transmit the load to the base plate, which, in turn, distributes it over the foundation. A face wall heavy enough to resist the earth pressure and live load transferred through the earth is placed in front of and built monolithic with the buttresses, and firmly anchored to it by means of reinforcing rods, usually with hooked ends. The face wall is continued beyond the bridge seats to form wings, and is supported at intervals by buttresses. The bridge seat consists of a heavy reinforced concrete slab resting upon the buttresses. A parapet wall at the back of the bridge seat forms the back or mud wall, as in a stone abutment. This wall has returns at the ends to the face walls. The construction, with the exception of the bridge seat and supporting buttresses, resembles closely the construction used for reinforced concrete retaining walls described in another chapter. It is desirable, if possible, to place the main buttresses directly under the girders, thereby eliminating bending in the slab forming the bridge seat. The buttresses or counterforts for the wing-walls are usually placed about 8-ft. centers. Ample reinforcement should be used to give all parts sufficient strength to resist any load which may be brought upon it. Plenty of anchor rods should be used to securely anchor the various parts together. The usual methods of analysis should be employed to determine the stresses in the various parts of the structure. The general features of rein-

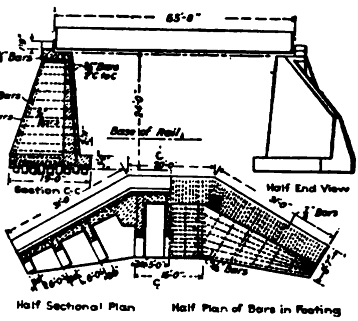


Fig. 684.—Abutment, C., B. & Q. R.R.

forced concrete abutment construction will be understood from the following examples.

C., B. & Q. Ry. Abutment.—Figure 684 shows plan, elevation and sections of a reinforced concrete abutment 24 ft. in height above the foundations built by the Chicago, Burlington & Quincy Ry. The size and arrangement of the reinforcing bars, together with the general dimensions of the abutment, are shown on the drawing.

K. C., M. & O. Ry. Abutment.—Figures 685 and 686 show plan, elevation and cross-sections of a 25-ft. abutment used on the K. C., M. & O. Ry. It should be noted that one wing wall is straight, while the other flares at an angle of 30°. The sizes and arrangement of the reinforcement are shown on the drawing, together with the general dimensions of the abutment.

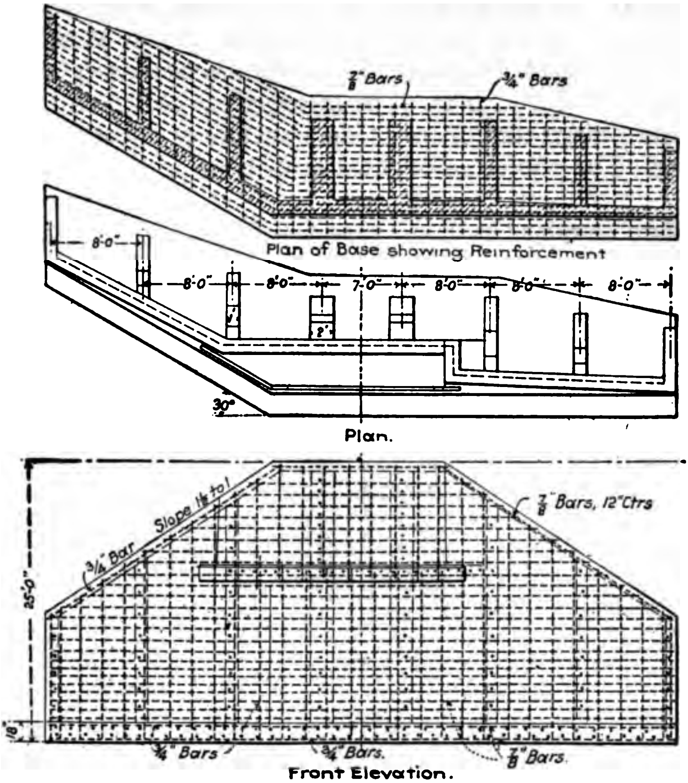


Fig. 685.—Plan and Elevations 25-Ft. Abutment, K. C., M. & O. Ry.

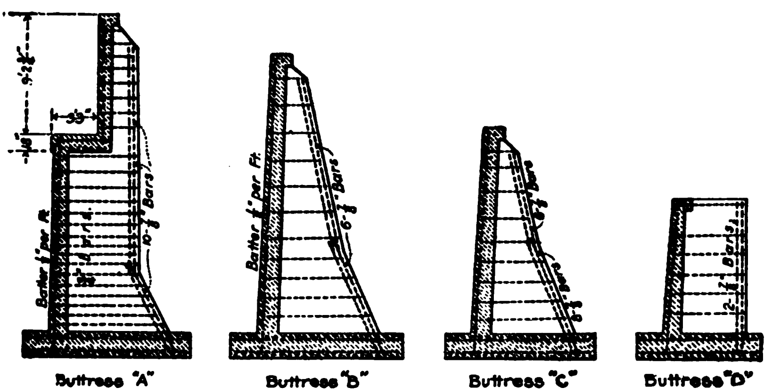


Fig. 686.—Sections of 25-Ft. Abutment, K. C., M. & O. Ry.

Wabash R. R. Abutment.—Figure 687 shows plan, elevation and sections of reinforced concrete abutment built at Monticello, Ill., by the Wabash Railroad. This structure has a height over all of approximately 32 ft. The arrangement of the reinforcement and general dimensions of the structure are shown on the drawing. The reinforcement used consists of ¾-in. corrugated bars.

Western Maryland R. R. Abutment.—Figure 688 shows detail

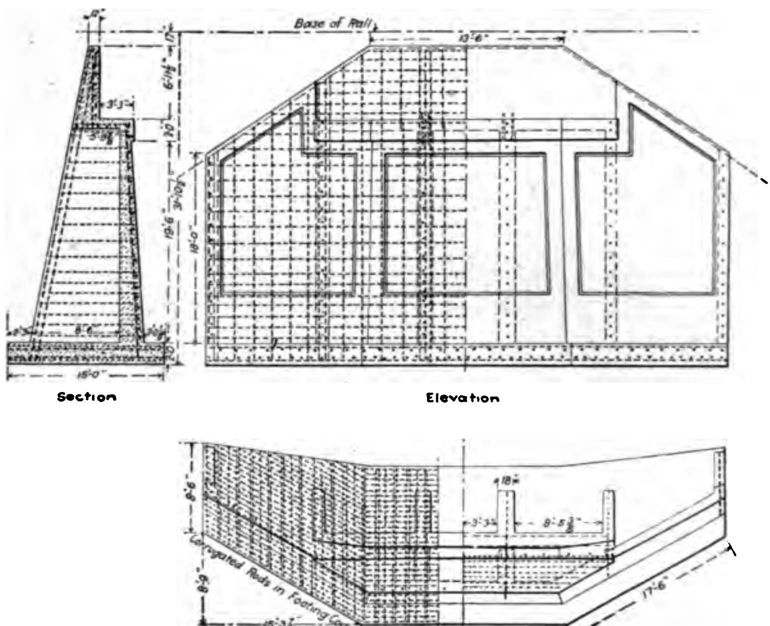


Fig. 687.—Abutment at Monticello, Ill., Wabash Ry.

of a concrete U-abutment, with an extreme height of about 43 ft. 6 ins., for the Western Maryland R. R. The parapet wall is 10 ft. 10¼ ins. high, and is 2 ft. wide at the top, and has a vertical front face. The bridge seat is 3 ft. 7 ins. wide, and extends the full width of the abutment. An offset of 4 ins. on the front and sides of the bridge seat walls 1 ft. 6 ins. below its top forms the edge of the coping. Other features of this abutment are clearly shown in the drawing.

Abutments for the Cairo Bridge, Illinois Central R. R.—A number of designs were prepared in 1903, under the direction of Mr. F. H. Bainbridge, for the high abutments at both the Illinois and Kentucky ends of the Cairo Bridge. Figure 689 shows original design for a wing wall abutment having wall flaring at an angle of 45°. The quantities of materials and estimated cost are as follows:

Concrete, 5,681 cu. yds., at $6	$34,086
Piles, 450, at $8.50	3,655
Total	$37,741

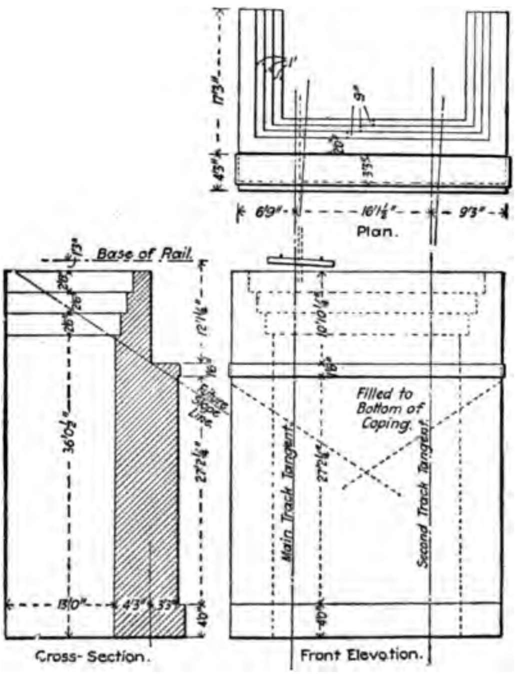


Fig. 688.—U-Abutment, Western Maryland R.R.

Figure 690 shows another design of an abutment having short, straight wing walls. On account of simple form work, the concrete in this design was estimated at $5.75 per cu. yd. in place. The quantities and cost are as follows:

Concrete, 3,353 cu. yds., at $5.75	$19,279.75
Piles, 480, at $8.50	4,080.00
Earth fill, 13,500 cu. yds., at 30 cts.	4,050.00
Total	$27,409.75

Figure 691 shows design for a U-abutment, with side walls connected by tie-rods protected by small concrete arches. The quantities and cost are as follows:

Concrete, 3,636 cu. yds., at $6	$21,816
Piles, 420, at $8.50	3,570
Earth fill, 4,000 cu. yds., at 30 cts.	1,200
Iron rods	50
Total	$26,663

Figure 692 shows design of hollow reinforced concrete abutment of the T-type adopted for the Cairo bridge. Side walls con-

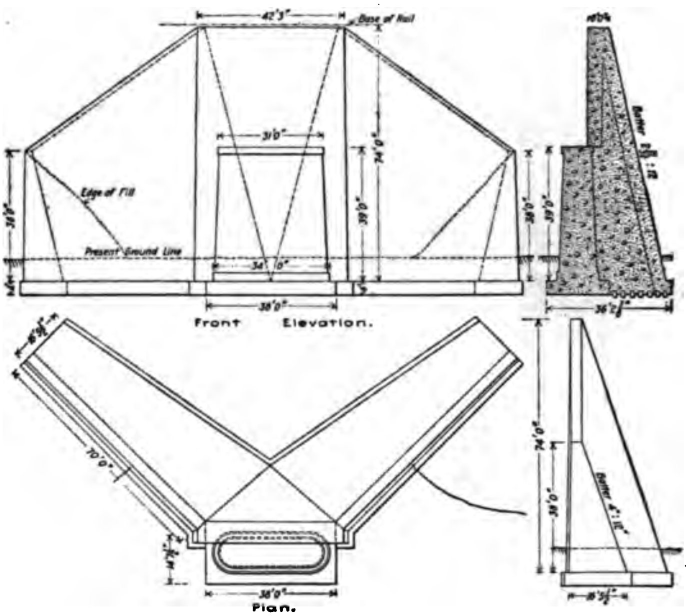


Fig. 689.—Wing Wall Abutment, Cairo Bridge.

nected by a longitudinal arch at the top, an inverted arch at the bottom, and an intermediate arch at mid-height form the main features of this design. The back of the arches are left open, allowing the sloping earth of the embankment to spill into the abutment, thus tieing it firmly to the bank, furnishing weight at the rear and preventing any forward thrust from the embankment. The horizontal partition serves for the same purpose, and prevents excessive earth pressures on the side walls. It is stated that the earth fill for this abutment is approximately the same

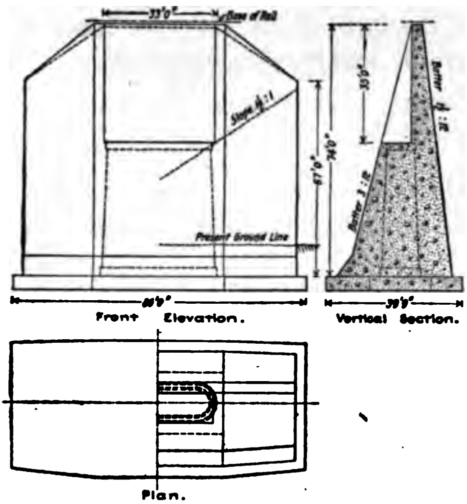


Fig. 690.—Design for Abutment, Cairo Bridge.

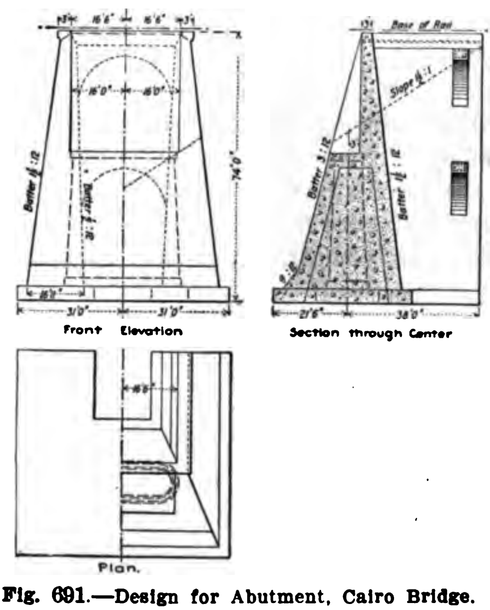


Fig. 691.—Design for Abutment, Cairo Bridge.

as for the original design with wing walls. On account of the extra work necessary for the construction of forms and richer concrete used, the price of concrete was taken at $7.25 per cu. yd. The quantities and cost are as follows:

Concrete, 2,660 cu. yds., at $7.25	$19,285.00
Piles, 337, at $8.50	2,864.50
Iron bars	300.00
Total	$22,449.50

An abutment of similar design was adopted for the Kentucky

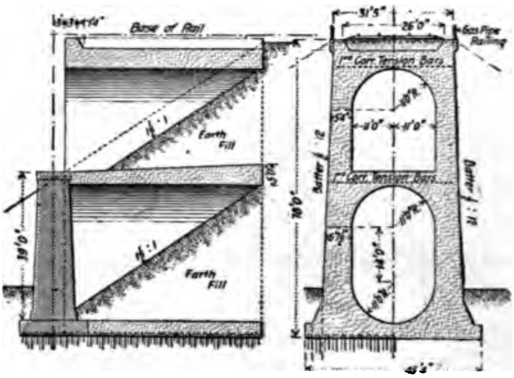


Fig. 692.—Design for Abutment, Cairo Bridge.

approach to the Cairo Bridge. The above abutment designs give some idea of various types of construction which may be used for high abutments. Similar or different types are adapted to the construction of abutments of less height. The difference in cost for the various types should be noted. The wisdom of making alternate designs before adopting any given design will be appreciated after comparing the above estimates.

CHAPTER XXXIV.

CONCRETE BUILDING BLOCKS.

Concrete building blocks, or cement blocks, as they are sometimes called, when properly made and used, form under many conditions an ideal material for the exterior walls of buildings. They are cheaper than brick or stone, and while somewhat more expensive than wood are much more durable, and possess the additional merit of being fireproof. They have all the advantages of brick and stone in point of comfort, coolness in summer, and warmth in winter. Building blocks are moulded hollow primarily to reduce their cost, but their hollow form makes them useful in other ways at the same time. The following are some of the advantages claimed for concrete blocks: (1) Their hollow form results in a saving of materials over brick or stone masonry, this often amounting to from 20 to 50 per cent. (2) The cost of laying concrete blocks is less than for brickwork. This is due to the fact that the blocks, being larger, have a much smaller number of joints, and require less mortar, and, being hollow, are of less weight than solid brick work. (3) A wall, properly constructed of good concrete blocks, is as strong or stronger than a brick wall of equal thickness. (4) Concrete blocks, being hollow, tend to prevent sudden changes of temperature within a house, making it cool in summer and easily heated in winter. For the same reason, if proper vents are provided, the interior of the walls will be free from the dampness so often met with in houses constructed of brick or stone. (5) The hollow spaces provide an easy means for running pipes and electric wires. These spaces may also be used wholly or in part for heating and ventilating flues. (6) Concrete blocks, being easily moulded, may often be used to replace cut stone, with considerable saving in cost. (7) The fireproof qualities of concrete are equal or superior to that of brick and stone. (8) The ease and rapidity with which a wall built of concrete blocks may be constructed make their use in many cases very desirable. Concrete blocks are constructed of cement, sand, gravel and broken stone. Lime is sometimes used to replace a portion of the cement. The fundamental principles governing concrete block construction, so far as relates to the qualities and properties of the materials, and the proportioning and mixing of the ingredients, does not differ materially from that of ordinary concrete construction, and has been fully discussed in the early chapters of this work. The mixtures used for blocks, however, differ somewhat from that used for other forms of construction. It is customary to use a dry mixture with many block machines in order to remove the blocks from the machine at the earliest possible moment. Good concrete blocks may be made with almost any machine, and even without the use of a machine. A concrete block machine enables the rapid moulding of blocks, and if any considerable number of blocks are needed will prove a paying investment. Concrete blocks may be obtained on the market in many parts of this country, and, if carefully inspected, may be safely used where it is undesirable for the builder to manufacture his own blocks.

Materials for Concrete Blocks.—Owing to its wide distribution and cheapness, sand is most commonly used for making concrete blocks. Gravel and broken stone mixtures give a denser mixture, and require less cement than sand mixtures. It is a mistaken idea that fine sand will give a denser mixture than sand and gravel or broken stone mixtures. In making building blocks the spaces to be filled are usually too narrow to permit the use of very coarse materials. Stone or gravel larger than 1 in. in size can seldom be used, and, generally speaking, sizes ranging from up to ½ or ¾ in. are the maximum sizes that will prove satisfactory. Where possible, graded sizes should be used, and for good results at least one-third of the material should be coarser than ⅛-in. Blocks made from such mixtures in the proportion of 1 to 5 will be found as good as a 1 to 3 mixture of sand alone. Generally, sand and gravel are the cheapest materials for block work.

Stone screenings, if they do not contain too much fine dust, will give as strong a block as can be made from sand and gravel. Screenings from soft stone should not be used, as blocks made from them are weak and undesirable.

Cinders are sometimes used for blocks with fair results.

Blocks made from them do not possess much strength, and are porous. For these reasons they should not be used in situations where great strength is required, nor should they be used for outside walls. They will prove the most satisfactory for interior walls carrying light loads.

Cement.—Natural and slag cements are sometimes used for block construction, and, while greatly inferior to Portland cement, may be used for work which will be constantly wet, but should be avoided for work exposed to dry air. The many excellent and reliable qualities of Portland cement make it the only satisfactory material for cement block construction.

Lime.—Lime is sometimes used to replace a portion of the cement. The most convenient form of lime for use in block-making is the dry-slaked or hydrate lime, now a common commercial article. No great saving results from its use, but its addition in the proportion of from one-quarter to one-half the cement used will cause the blocks to set more rapidly, make them lighter in color, and make the concrete denser and more impervious to moisture. In sand mixtures with the proportion of 1 to 4 or 1 to 5, at least one-third of the cement may be replaced with slaked lime without loss of strength.

Proportioning.—It is customary to fix arbitrarily the proportions of materials used for cement blocks. When sand is employed for the aggregate, the proportions commonly used vary from 1 : 3 to 1 : 6. When broken stone and gravel with sand are used for the aggregate, leaner mixtures will give equal strength. Thus a combination of 1 part cement, 3 parts sand and 6 parts broken stone will prove satisfactory if the stone is of properly graded sizes. Another mixture commonly used is 1 part cement, 2 parts coarse sand, 2 parts fine gravel and 2 parts coarse gravel. Mixtures commonly used for ordinary concrete construction, viz., 1 : 2 : 4 and 1 : 3 : 5, will prove satisfactory. Care should be taken to secure as dense a mixture as possible. Chapter IV. of this work should be consulted as to the method of determining the best mixture with the given materials. When facing is used, mixtures in proportions varying from 1 to 1 to 1 to 3 are used for the face mixture, and 1 to 5 or 1 to 6 mixtures for the backing. While the poorer mixtures given above will possess ample strength, they will be porous. It is believed that a 1 to 3 or

1 to 4 sand mixture and a 1 to 5 sand and gravel mixture are about as poor as should be used. The addition of hydrate of lime tends to make the block less porous. Plenty of water used in mixing also tends to reduce the porosity of the block.

Proportion of Water.—The amount of water used in mixing affects greatly the quality of the block. Dry mixtures are necessary for rapidity of moulding, as only when they are employed can the block be readily removed from the machine. If blocks are made from too dry concrete, they will remain soft and weak no matter how thoroughly they are sprinkled afterward. On the other hand, too much water will cause them to stick to the machine plates, and they will not retain their shape if removed at once from the moulds. It is possible to obtain a concrete of such consistency that the block may be removed from the mould at once and still possess first-class hardening properties. A practical method of determining the proper consistency is to squeeze some of the mixture in the hand, and see if its holds its shape upon relieving the pressure; if not, it is too dry. It should hold together when placed on the other hand, and should not wet, but slightly moisten, the hand when squeezed in it. No fixed percentage of water can be specified, as the character of the materials, the moisture in the atmosphere, as well as other causes, govern the amount of water needed.

For blocks moulded by pressure, a medium wet mixture is used. The pressure causes the surplus water to flush to the surface. For poured work a mixture so wet that it is fluid is used. Castings in sand moulds are made by the use of a very liquid concrete; fine limestone screenings prove the most satisfactory aggregate for poured work, as sand and gravel settle too rapidly for such thin mixtures.

Mixing and Depositing.—As in all other concrete work, thoroughness in mixing and care in depositing are necessary to success in cement block manufacture. Chapters V. and VI. should be carefully read in this connection.

Curing.—The process of curing after the block has been moulded is the most important in the manufacture of concrete blocks. All blocks manufactured by the medium wet or medium dry process should be made under cover, and protected from the sun, from dry currents of air, and from frost until thoroughly cured. The block should remain on the pallet at least for from 24 to 36 hours. As soon as the concrete has sufficiently set so that the water will not wash the surface, usually from 4 to 12 hours after moulding, the blocks should be gently sprinkled. From this time they should be sprinkled two or three times daily for from 7 to 14 days. Plenty of water is necessary to the process of setting, and sufficient should be used to supply moisture to all parts of the block. Blocks should never be allowed to dry out on the surface until the center has thoroughly cured. The moment that the surface of the block begins to turn white it is a sure sign that it needs water. Too much water can not be used. Care should be taken to so pile the blocks that they will receive moisture equally on all sides. The slower the drying of the block, the harder and tougher will it become. A sand floor kept wet may be used to insure a damp atmosphere. If a suitable shed or building is not available for curing, the blocks may be covered with canvas, burlap, hay or other covering which will retain the moisture, and at the same time keep dry air from circulating around the blocks. It is essential that as near as possible the conditions under which the blocks are cured should be uniform.

An advantage claimed for one of the face-down processes in which pallets faced with concrete are used, is that, while setting, the face of the stone is protected from washing and from drying out too fast. By this process the blocks are claimed to be self-bleaching, being practically cured with the face in water, as the face plate is waterproofed and concave. When the block is sprinkled, the water flows down into and is retained in the face plate. It is claimed that by this method the face of the block is bleached to an almost pure white, while when made with the same materials under ordinary conditions it would be gray.

In winter the blocks should be protected from freezing for at least 10 days after moulding.

For curing a dry mixture will require more water than a medium or wet mixture, and should be kept in a moist atmosphere for a longer time than the latter. In the former case blocks should be kept thoroughly moist for at least 20 days, while in the latter case about 10 days will be sufficient.

Steam has been used for curing blocks in a number of instances, and, it is claimed, with success, 48 hours hardening the block sufficiently for it to be placed in the wall. Tests on blocks cured in this manner are too few to draw any definite conclusions, but it is probable that the most satisfactory results will be obtained if the blocks are first allowed to harden 24 hours in moist air before subjecting them to the steam bath.

Facing.—The use of a thin facing of different materials from the mixture used for the body of the block is growing in favor. The objects of facing are threefold, viz., a saving in cost of materials, the securing of a dense and impervious facing, and an improvement in the appearance of the exposed surface of the block. The saving in materials results from the use of a coarse, lean mixture for the body of the block, and a fine, rich mixture of only from ¼ to ½ in. in thickness for the face. This facing is usually made of a mixture of cement and fine aggregate, varying in proportions from 1 to 1 and 1 to 3. By a judicious selection of the sand or crushed stone used for the facing, a pleasing surface may be obtained. By this means, within certain limits, any desired color may be obtained. To secure permanent results colored sand or stone should be used, but the addition of coloring matter to the facing mixture will produce a similar result. In the latter case, however, there is danger of the colors fading, and if too much coloring matter is added the strength of the concrete may be affected. Chapter VIII. should be consulted in this connection.

In the manufacture of blocks with a facing coat only those methods should be used in which the body of the block is placed at the same time as the face, and the two thoroughly united by tamping or pressure. Two methods are usually employed in placing the facing concrete. In the first it is placed and held in position by a backing board; the back concrete is then put in and tamped as the backing board is withdrawn. In this method there is danger of a cleavage plane being formed between the facing and backing concrete. In the second method what is known as a face-down machine is used, the facing concrete being placed on the face plate, tamped, and the backing concrete then placed upon it and tamped. The two mixtures are by this method thoroughly united. A third method is employed in pressure machines, the facing material being placed last, and the pressure plate is the facing plate of the mould.

The form of faces most commonly used are smooth face, bevel-edge, corrugated or ribbed, pitch or rock face, and special designs of ornamental faces. Much taste and judgment should be used in the selection of facing to be used. It is believed that the use of the simple forms and an avoidance of a repetition of a fixed pattern of ornamental design will give more artistic results. It should be remembered that the concrete block or cast stone is a material that possesses ornamental and artistic properties *per se*, and the mistake avoided of trying to imitate other kinds of stone. Appearances which are unnatural to concrete should

Fig. 693.—Example of Plain and Ornamental Concrete Blocks.

be avoided. It is believed that smooth, ribbed or paneled surfaces with good ornamental patterns for friezes or cornices will give tasty and artistic results. It should, however, be remembered that the smooth, dead look which rich cement mixtures have are not pleasing, and that a rough, varied surface is more pleasing to the eye. The methods of treating concrete surfaces described in Chapter VIII. should be considered in this connection.

Some of the possibilities of the variety which may be secured in concrete-block ornamentation are shown in Fig. 693.

Examples of Block Construction.—Some of the architectural features of concrete block construction are shown in Figs. 694 to 697.

Figure 694 shows a popular style of porch work made entirely of moulded concrete. Figure 695 shows a small resi-

Fig. 694.—Example of Porch Work in Concrete Blocks.

dence at Albuquerque, New Mexico, built of cement blocks. This kind of construction will prove both cheap and durable in many localities where the cost of other kinds of buildings is

Fig. 695.—Concrete Block Residence at Albuquerque, N. Mex.

prohibitive. Figure 696 shows a somewhat more pretentious residence at Newark, Ohio; while Fig. 697 shows a court-house built entirely of blocks and artificial stone.

Fig. 696.—Concrete Block Residence at Newark, O.

Fig. 697.—Example of Concrete Block Public Building Work.

Shape of Blocks.—The earliest form of hollow blocks used in this country are those shown in Fig. 698. This block is known as the Palmer block. As will be noted, it is simple in form, having a transverse web at either end and two webs near the middle, so that half blocks can be formed without changing the cores in the machine. As will also be noted, an L-shaped corner block is used. This block has been widely copied and many other forms devel-

Fig. 698.—Examples of Harmon S. Palmer's Hollow Concrete Blocks.

oped from it. As originally designed, the air space varied from 20 to 33 per cent. In modern block construction, however, the percentage of air space has been greatly increased, so that now it is not unusual to have an air space of from 50 to 55 per cent. of the horizontal section of the block.

The block shown in Fig. 699 has a single transverse web at the middle, and is the form made by the greatest number of machines

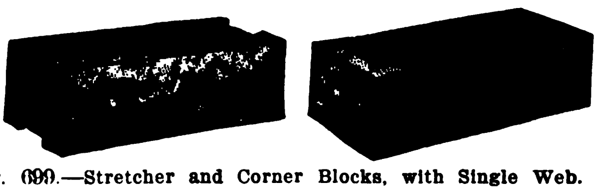


Fig. 699.—Stretcher and Corner Blocks, with Single Web.

at the present time. Fig. 700 shows another favorite form made with a single core. The reduction in the number of transverse webs reduces the liability of penetration of moisture in case of heavy rains. Again, it is easier to tamp around one core than two or more, and a more dense block with the same amount of care and labor in tamping will be secured. There is also less danger of injuring the block by the withdrawal of the cores.

The desirability of securing perfect insulation and preventing the penetration of moisture through the cross ribs has brought about the development of two forms of blocks, viz., blocks with a staggered air space and the two piece block.

Fig. 701 shows a one piece block with staggered air space.

Fig. 700.—Example of National Hollow Block.

This is known as the Miracle block. The arrangement of the webs and hollow spaces is such that each web will be backed by an air space and no portion of the solid concrete extends directly from the outer face of the wall to the inner side. This arrangement makes the penetration of moisture from the face to the rear

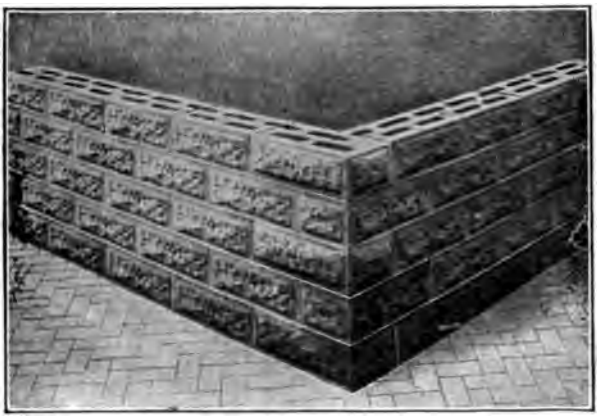


Fig. 701.—Miracle Block with Staggered Air Spaces.

of the block practically impossible. It is stated that this arrangement has proved entirely satisfactory. Another block of the same type with a multiple air space is shown in Fig. 702. In this block, however, the arrangement of the air spaces is somewhat different.

Fig. 703 shows a block of the two piece type. This block is manufactured by the American Hydraulic Stone Company's system. When sufficient care is taken to secure a good bond this block will give a strong and satisfactory wall. The bond is secured by lapping the short and long arms of the T's in adjacent

Fig. 702.—Reid Hollow Block, with Staggered Air Spaces.

courses. As this block has only one face, interior cores are eliminated and it is possible to make the blocks by means of mechanical or hydraulic presses. By the use of this block multiple air spaces may be secured, as shown in Fig. 704. Fig. 705 shows an

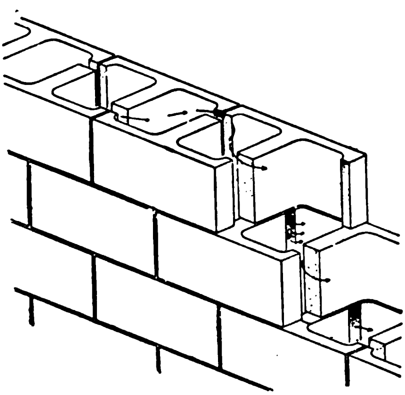


Fig. 703.—American Two-Piece Block Hollow Wall.

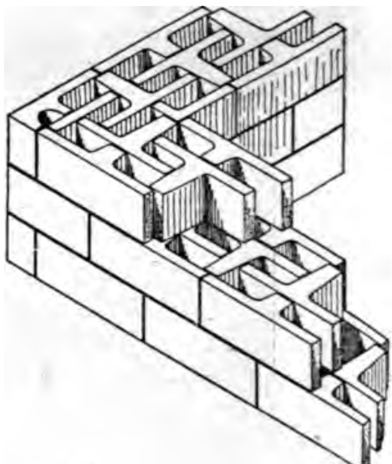


Fig. 704.—Wall with Multiple Air Spaces.

application of segmental blocks of this system to the construction of circular stacks or bins.

Another block used to secure a wall with a multiple air space is shown in Fig. 706. This block is known as the Mandt block.

Many blocks varying somewhat in form from those given above are manufactured, but it is believed that the above examples illustrate the principal types now in use.

Processes of Manufacture.—Three general processes are used for the manufacture of concrete blocks, by tamping, pouring or pressing. The most common method is that in which a dry mixture is tamped into the moulds either by hand or by means of pneumatic machinery. In this process a dry mixture of sand and cement is deposited and gradually tamped into the moulds. Light and frequent ramming is necessary to thoroughly compact the materials, and only by this means will the quick adhesion of the particles result, which is necessary in order that the mass remain

Fig. 705.—Two-Piece Blocks Used for a Circular Stack.

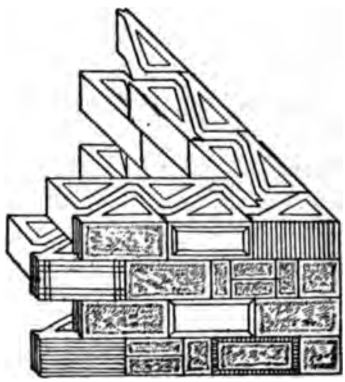


Fig. 706.—Mandt Two-Piece Block Hollow Wall.

firm when removed from the moulds. For the last two or three years an effort has been made to use a light pneumatic hammer for the purpose of tamping. This reduces the labor and secures a uniform product. When the tamping is done by hand the quality of the product will be uncertain unless great care, industry and endurance are exercised to secure uniform results.

The process of pouring has already been discussed in a previous chapter. Either iron or sand moulds are used. A wet concrete is employed, it being of such a consistency that it will easily flow into all the interstices of the mould. As it requires several hours for wet concrete to set sufficiently to retain its form, a large number of moulds will be necessary when this process is used. On this account it is seldom used for ordinary block work, but is more particularly adapted to the manufacture of ornamental forms.

During the past two or three years the manufacture of blocks by means of pressure, either mechanical or pneumatic, has been undertaken by a number of concerns. The method consists essentially of compressing in moulds, which are properly vented for the escape of air, a medium wet mixture of coarse concrete. An aggregate graded in size up to ¾ or 1 in. diameter for the largest size stone is usually employed. Either hydraulic or mechanical pressure is applied as soon as the moulds are filled, the object being to apply a uniform pressure to all parts of the block. It is essential that all blocks be subjected to the same pressure if a uniform product results. Two methods are employed to secure blocks of uniform density, first by applying the same pressure to all blocks, the pressure being read by means of a gauge, and second, by using exactly the same amount of materials for each block and stopping the pressure when the pressure plates have reached a point to give a block of the desired size. In the first method, blocks of variable size result, while in the second a perfectly graded and mixed aggregate must be used. It is practically impossible to secure ideal results by either method, but by exercising great care fairly satisfactory results will be obtained by either method. Both methods have been employed for the manufacture of asphalt paving blocks for a number of years. A study of the resulting product, as well as the processes, by the author, shows that this method of manufacture has not as yet reached the state of perfection to be desired. The difficulties experienced in the operation of machines of this type has caused at least one large manufacturer of block machinery to abandon entirely the use of pressure machinery.

Block Machines.—The process of manufacture determines the character of machine to be used for block construction. In general there is a similarity among all machines used for manufacturing blocks by tamping a dry mixture composed of a comparatively fine aggregate. These machines consist essentially of a set of moulds and cores with or without a metal stand or platform to support them in a position convenient for performing the operation of depositing, tamping and removing the block. Many mechanical contrivances differing somewhat from each other are added to the simple moulds, cores and stand or platform to facilitate and reduce the cost of operation. Again, provision must be made for giving variety to the exposed face of the block, and, when desired, to make possible a variation in the mixtures used for the facing and body of the block. The mechanical operation must also be such that the finished block can be easily and quickly removed from the machine without injuring the green concrete.

Cement blocks have been successfully made by means of wooden moulds of extremely simple design. When a quantity of blocks are needed it is believed that the purchase of one of the better types of machines will prove a good investment. Machines

Fig. 707.—Palmer Block Machine Ready for Depositing the Concrete.

of this type are divided into two classes: those with a vertical and those with a horizontal face plate. In the former, the facing plate mould stands vertical and the block is simply lifted from the machine on its base plate as soon as tamped. In the other type the face plate forms the bottom of the mould, the cores are withdrawn horizontally and the block with its face plate tipped up in a vertical position for removal. When it is desired to use a facing mixture, it is necessary to provide a parting plate in the vertical facing machine, while with a face down machine the operation is very simple, consisting simply of depositing the facing coat and then tamping the backing concrete upon it. Fig. 707 shows the latest model of an upright machine made by one of the first manufacturers of block machines in this country. The machine is shown closed and ready for depositing and tamping in the material. Fig. 708 shows the machine open and ready for the removal of the block. The cores, which are withdrawn downward, are shown below the platform upon which the block rests.

Fig. 708.—Palmer Block Machine Ready for Removing Block.

Fig. 709.—Face-Down Machine, Closed.

Fig. 710.—Face-Down Machine, Open.

A machine of much simpler construction but of the face down type is shown in Fig. 709. The sides of the mould are locked in place and the machine ready to receive material. Fig. 710 shows the face tipped up and the block resting upon the pallet ready for removal.

Figs. 711 and 712 show a machine which is removed after the block has been moulded without disturbing the green concrete block. When this machine is employed it is not necessary to handle the block until it has hardened. Fig. 711 shows the machine ready for the mixture, while Fig. 712 shows the manner of releasing of the finished block.

Figure 713 shows a machine of the mechanical pressure type which has been extensively used in the manufacture of two piece blocks. Usually mechanical or hydraulic pressure machines are of heavy and complicated construction.

Fig. 711.—Pettyjohn Machine Ready for Depositing the Concrete.

Fig. 712.—Moving the Mould Instead of the Block.

When the pouring process is used, sand moulds, moulds of wood, cast concrete or metal are employed. The methods of securing the desired impression in the mould has already been discussed in another chapter. Where wooden moulds are employed they should be waterproofed to prevent the wood from warping. This method will hardly become popular for ordinary block construction on account of the large number of moulds necessary for any considerable output.

Building Construction.—As with other types of construction, a good foundation is necessary. The foundation walls may be made of blocks as well as the upper walls. These should be somewhat heavier than those used for the first story walls. The thickness of walls usually employed for block construction are as follows:

For one-story buildings	8-in. walls.
For two-story buildings	10-in. walls.
For three-story buildings	10 and 12-in. walls.
For four-story buildings	10, 12 and 15-in. walls.

The thickness of the walls are, of course, increased from the top downward.

Fig. 698 shows method of supporting floor beams by means of notched blocks. Metal hangers are sometimes used as shown in Fig. 714. Either of the above methods will prove satisfactory.

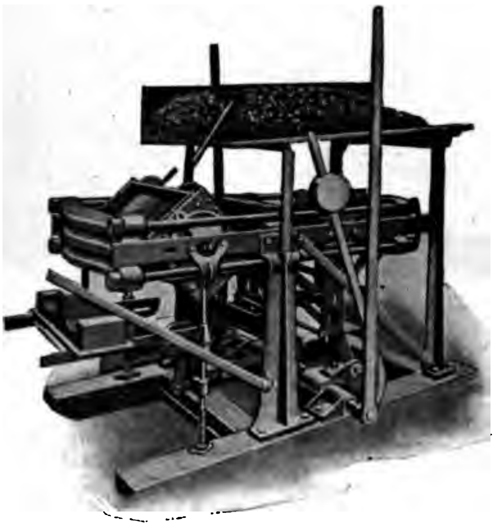


Fig. 713.—American Hydraulic Stone Pressure Block Machine.

As it is impossible to drive nails into concrete, some means must be employed for fastening door and window trim, base boards, etc. A metal plug which is laid in the mortar joints, as shown in Fig. 715, or wooden plugs cast in the concrete may be used for this purpose.

Where well made blocks are employed, plastering may be done direct upon the inside face of the blocks. As the wall face is quite smooth, the scratch coat may usually be omitted and the hard coat applied directly upon the inner face of the wall.

By the use of special moulds, sills and lintels for windows and doors may be cast in the same manner as ordinary blocks. If this is not done, they may be built in place in the same manner as in reinforced concrete buildings.

Mortar joints for cement blocks vary from ⅛-in. to ¼-in. in thickness. The smaller area and thinner joint, as well as reduction

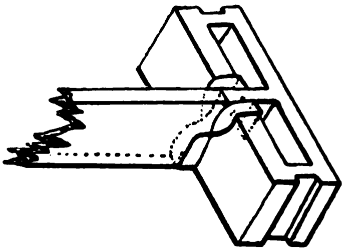


Fig. 714.—Stirrup for Supporting Floor Beam.

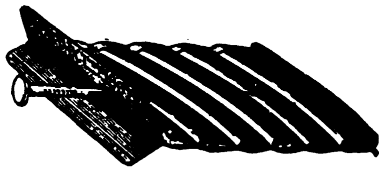


Fig. 715.—Metal Plug for Nailing.

in number of joints, results in a great saving in mortar over that necessary for brickwork. A good cement mortar for laying blocks may be made as follows:

Portland Cement.	Sand.	Lime.
1 part	2 parts	½ part
1 part	3 parts	1 part

Mix to a plastic state but not too thin, as it weakens the mortar.

Cost of Concrete Building Blocks.—Much misinformation has been given by makers of concrete block machinery as to the cost of concrete blocks in their desire to sell their machines. Data from trial tests under unusually favorable conditions should not be used to determine the cost of manufacturing concrete blocks when a comparison is desired with that of other and older methods. With equal costs, however, the hollow block possesses many desirable qualities which should lead to its adoption. It will generally be found, however, that a hollow block wall will be cheaper than either brick or stone walls, and in some instances nearly if not quite as cheap as frame construction. The costs given by a number of manufacturers for a single block varies from 7 to 20 cts., the highest price, 20 cts., being for a block which will lay 2 sq. ft. in the wall or a cost of 10 cts. per sq. ft. of wall. Assuming the price of brick at $8.00 per thousand, the cost of materials alone for 1 sq. ft. of wall 12 ins. thick will be

16 cts. The cost of laying brick varies, but may be as much as $8.00 per thousand, although in localities free from the exactions of labor unions the cost may be less than half this amount. From 4 to 5 cents per block is given as a fair price for laying cement blocks.

It should also be remembered that a hollow concrete wall built of moisture proof blocks does not need furring strips or lathing, but the plastering is applied directly upon the inside of the wall. This results in an additional saving. Thus a considerable saving may result by the use of concrete block construction in place of brick work. This will amount at times to from 20 to 50 per cent., not to mention that a dryer and warmer building is secured.

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